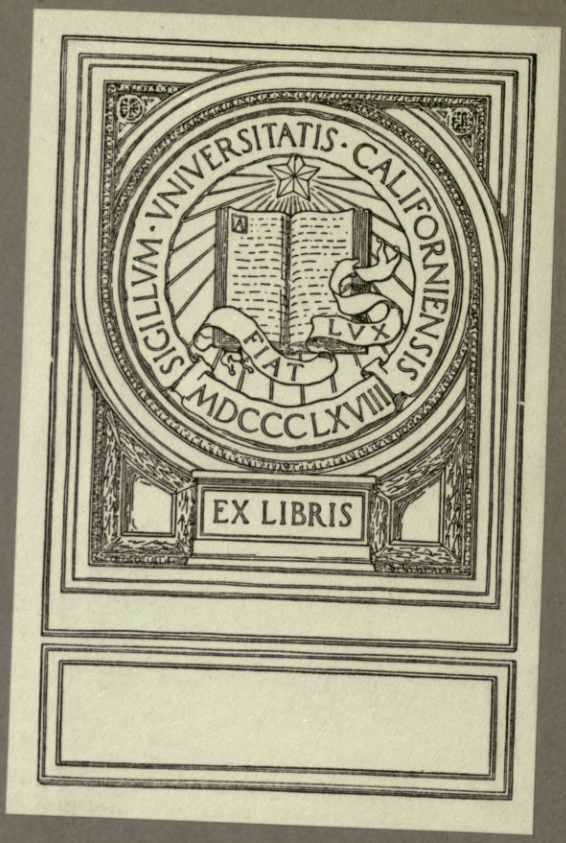
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CASSELL'S REINFORCED CONCRETE

Cassell's
Reinforced Concrete

A Complete Treatise on the Practice and Theory of Modern Construction in Concrete-Steel

Edited by

Bernard E. Jones

Editor of "Building World"

Assisted by

Albert Lakeman

M.C.I., Honours Medallist in Building Construction

and by a Staff of Specialist Writers

Illustrated by 171 Photographs and about Five Hundred Diagrams and Working Drawings

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Preface

THE purpose of this book is to provide a practical, simply-worded guide to construction in reinforced concrete, as distinct from the theoretical and often obscure treatises that have appeared in large numbers during the past few years. The theoretical side of the subject has not, however, been overlooked, as any treatise on reinforced concrete that ignored methods of calculating the various members would be obviously incomplete, especially as this method of construction offers great possibilities of scientific design. As a matter of fact, the two theoretical chapters are a strong feature of the book, having been written, at my request, by Mr. Albert Lakeman, who has brought a trained mind to bear upon his subject, and, in addition to explaining important principles in a simple and concise manner and clearly showing the practical application of the theories he expounds, has imparted to his text the unusual element of freshness. Mr. Lakeman has wisely begun at the beginning, and, whilst he could not finish at the end—for one thing, the end is not yet—he will be found to take an apt and diligent student far enough on the road for him to be able to find the rest of the way for himself; and, after all, no book can do more than that for anyone.

The book is largely concerned with the practical side of reinforced concrete—with what has been done and can be done in the compound material, and how to do it. The introductory chapter having explained the advantages of reinforced concrete, an historical chapter (every definite date in which I have verified from official records) puts the reader in possession of a variety of facts, many of which will throw light on the subject as his knowledge of it grows. Concrete and steel, as materials, are discussed in the two following chapters, in the earlier of which special attention is devoted to portland cement and the modern methods of mixing and handling concrete. Then come the explanations of theory, followed by two chapters respectively devoted to the methods employed in erecting a building and the forms and centerings necessitated in general reinforced concrete work; these two chapters are noteworthy, I think, from the practical character of the information they give and the very large number of explanatory drawings and photographs which they include. Concise descriptions of the chief commercial systems follow, and later chapters deal respectively with architectural and surface treatment, durability, waterproofing, arches and bridges, and quantity surveying, estimating, measuring and pricing. The concluding chapter describes and illustrates a number of works carried out in reinforced concrete, varying from a palatial club-house and commercial "sky-scraper" to railway sleepers and sewer pipes.

Every endeavour has been made to render the volume as complete as possible, in the hope that it will be useful to all classes of readers, both as a text-book and as a work of reference.

I have been happy in having the co-operation of a number of experts (a list of

whom is given elsewhere). Each of these writers has contributed or revised the section relating to the particular phase of the subject he is most competent to deal with; and from many of them I have received valuable suggestions which I have been able to apply. I wish to acknowledge, also, the kindly treatment accorded me by a large number of firms identified with special systems, materials, machines, etc., for, in a number of cases they have supplied me with information, drawings, and photographs. Additional thanks are due to Messrs. Geo. West and Co. for permission to make brief extracts from the historical notes published in a "Lock-woven Mesh" handbook.

I owe a special debt of gratitude to the premier concrete journal, *Concrete and Constructional Engineering*, which has placed many facilities at my disposal and has been good enough to lend me a large number of blocks. To the files of that journal, too, I have gone for help in many instances when other sources of information proved vain.

The illustrations to this work include no less than 171 photographs and about five hundred diagrams and working drawings; and the large number of "progress" photographs, showing the actual practice of reinforced concrete work, constitute, I hope, a specially valuable feature of the book, and considerably enhance its teaching value.

B. E. J.

La Belle Sauvage,
London, E.C.

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Reinforced Concrete

Introduction; What Reinforced Concrete Is

The Limitations of Concrete.—For thousands of years concrete has been known and used as a building material possessing many valuable properties—universality and consequent cheapness; ease of handling, placing, and shaping; ability to resist fire, water, and other destructive influences; and great strength under compression. (A body, itself evenly and rigidly supported, is under compression when a load is placed on it tending to squeeze it.) Until the twenties of the nineteenth century, concrete was made by mixing together stones, gravel, etc., and lime, but such was the weakness of the material in tension (that is, when subjected to a force that tended to stretch it or bend it) that its employment had to be restricted to such applications as foundations, relatively thick walls, and the like. On the introduction of portland cement in 1824, the quality of concrete, particularly as regards the tensile strength, very greatly improved; and modern cement manufacture—a highly specialised industry—has produced a material with an appreciable ability to resist a stretching force. But good as the best cement is, it is still not good enough for use in a structure by itself except under compression.

Reinforcement.—Fortunately, it early occurred to some ingenious workers that if they could use with it a second substance which would supply the tensional strength lacking in the concrete, they would then be in possession of a well-nigh ideal material for structural purposes. Tile, wood, bronze, iron, and steel have all been tried, and the preference goes to the last-named. Modern reinforced concrete, then, is simply a combination of a material (concrete) strong in compression but weak in tension with one (steel) that is itself strong in compression, but so much more strong relatively in tension.

It may occur to some readers to ask why steel should be used to bolster up a deficiency in another material. Why, it might be asked, is not the steel used alone, and full advantage taken of its valuable qualities? Steel-frame construction was earlier in the commercial field than reinforced concrete, but it has not prevented the latter system from forging ahead and making a truly notable progress. There must be a good reason for that, and it is to be found in a number of considerations. Steel is relatively expensive; the minimum quantity is used in reinforced concrete construction. A steel structure involves the making of thousands of riveted joints, and calls for a large amount of skilled work; reinforced concrete construction is monolithic (literally, "as one stone"), and there is practically no jointing. Steel is corroded by atmospheric action unless immediately protected, and the protection needs periodic renewal; concrete is scarcely affected by atmosphere, requires no protection and no maintenance, and it preserves the steel even brighter than when it was inserted; indeed, steel soon loses a coating of rust when embedded in concrete. Most important of all, an unprotected steel structure is the worst possible for resisting fire, by reason of the metal being such a good conductor of heat; the steel rapidly expands as the temperature rises and contracts again as the cold water from the hose reaches it, thus pushing and pulling apart the elements of the structure and often causing complete ruin; then its softness, should an extreme temperature be reached, robs it of its strength, and the steel girders become bent and twisted into a mere entanglement. On the other hand, concrete is a poor conductor of heat, and therefore well protects the embedded steel and localises any ill effects. Reinforced concrete possibly brings the goal of an absolutely fireproof structure within the attainment of our own time.

Terminology.—Reinforced concrete, in the commercial acceptance of the term, is

concrete in which steel rods have been embedded to increase its strength. The term is somewhat vague, but no completely satisfactory substitute for it has been suggested. The material was (and often is) known on the Continent as "Monier concrete," but this does not explain itself, and may therefore be dismissed. "Ferro-concrete"—one of the best terms yet introduced—has been monopolised as a trade name, although it is used as a generic term in technical literature; even that designation, though, is not strictly correct, since "ferro" of course, is formed from "ferrum" or "fer" (respectively Latin and French for iron); but iron, as a structural material, has long given place to steel, made by "alloying" iron with carbon. The term "reinforced concrete" does not, unfortunately, suggest the material employed with the concrete; for example, at an early date concrete was reinforced with wood; indeed, wood-reinforced concrete, known as "ligno-concrete," is now attracting some attention, but the term under discussion does not differentiate between "ferro-concrete," "ligno-concrete," or "æsso-concrete" (bronze-concrete), the earliest type of reinforced concrete known. The term is open to another objection; in a sense, the steel is itself just as much "reinforced" as is the concrete, and therefore "reinforced steel" might seem as logical a designation. The term "concrete-steel" has its advocates, amongst whom we beg to take our place; but it has not caught the imagination to the extent that some of the other terms have. Much the same can be said of "armoured concrete." It appears that the one term—other than trade names—likely to become universal is "reinforced concrete," and for that reason it has been adopted as the title of this book, the more precise "concrete-steel" finding a place in the sub-title.

A Compound Material.—Concrete-steel may be regarded as a compound material, and not as two distinct materials. The steel is wholly embedded in the concrete, and the adhesion between the two, after the maturing of the concrete, is such that a force sufficient to pull the bars from the concrete would require to be 500 lb. to 600 lb. per square inch of surface contact, although it is regarded as safe to allow only 100 lb. per square inch. Further, the two materials may be considered as expanding and contracting at the same rates. The coefficient of expansion of concrete is ·000006 (per 1° F. change of temperature), and that of steel averages ·0000066; the difference is therefore only 1 in about a million and three-quarters. In spite of all this, there exists a division in the ranks of the concrete-steel engineers and specialists. On the one hand are those who believe that the natural adhesion of the concrete to the steel is sufficient to answer all ordinary requirements. On the other are those who say that experience teaches that this adhesion ought to be assisted by a mechanical bond, and they therefore "deform" the bar to remove the least likelihood of its slipping when subjected to severe tension, this being a time at which the sectional area of the bar is liable to be reduced, thus lessening or destroying the adhesion. Deformed bars are also used to allow of the employment of a steel having a greater tensile strength than mild steel. The chief Continental systems use plain bars, while the chief American systems use bars of special shape. This subject suggests other considerations, which would, however, lead us too far into the theory of the matter, and which will therefore be relegated to a later chapter.

Comparisons.—Stated in approximate terms, the compressive strengths of concrete and steel are as 1 : 28; the tensile strengths as 1 : 280; and the weights, bulk for bulk, 1 : 4. To these ratios may be added those of cost, bulk for bulk; let us assume the approximate costs of good concrete (materials only) and steel to be 1 : 80, although this of necessity will vary. Working on these data, some interesting comparisons may be made. Figs. 1 to 3 show sections of circular columns, all assumed to be of the same height and capable theoretically of supporting the same weight. The concrete column must be 28 times the area in cross-section of that of the steel column, and nearly 1½ times that of the concrete-steel one; roughly, the relative areas would be about: steel, 4; plain concrete, 112; reinforced concrete, 80. This comparison does not adequately show the true advantage of reinforced concrete, since the reinforcement in a tall concrete column is absolutely necessary, the rods and hooping preventing the concrete from bursting under a heavy load. Again, in the case of eccentric loading, one side of the column may be in tension, and steel is then required to take this stress. The relative sizes will depend upon the

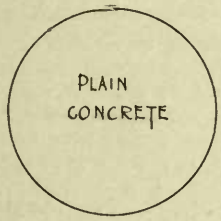
percentage of reinforcement, and the foregoing is based upon the minimum usually employed. In the case given, the area of steel in the steel column is twice that in the reinforced column, and it is this material that is the more expensive of the two.

Again, the weight of the concrete column is 7 times that of the steel one, and about 1·3 times that of the concrete-steel; but the costs (for materials alone) will be approximately in the proportions of 7, 20, and (nearly) 15.

Now consider three beams, respectively of concrete, steel, and reinforced concrete, of the same depth and of the same value of resistance—that is, capable of supporting the same load. Figs. 4, 5, and 6 are cross sections of these beams, *d* indicating depth (the same in all three cases) and *b* breadth. Expressed in inches, the concrete beam has a breadth of 76 in., the steel beam ·285 in., and the reinforced concrete beam 8 in. Their relative weights are (roughly) 70, 1, and 7½. this might seem to show an advantage for the steel beam, but it will be noted that the steel beam contains nearly five times as much steel as the reinforced concrete beam, the relative costs (material only) working out as (roughly) 6·2, 1·7, and 1. This economy of steel is possible because *all* the steel in the reinforced concrete beam is placed as far as possible from the "neutral axis," and can therefore be stressed and utilised to its full value; whereas in the steel beam, much of the metal is adjacent to the neutral axis, and part of its value is lost. (When a beam is loaded reasonably, it tends to bend, and its length is altered, but there is a layer of fibres or a plane, the length of which remains unaltered; in a cross-section this "neutral surface" is, of course, a line, which is known as the "neutral axis.")

It will be understood by the practical reader that the diagrams to this chapter are purely comparative, as steel is not commonly used in the form shown. If for the rectangular steel beam were substituted a rolled steel joist having an equivalent moment of resistance, its area would exceed 3½ sq. in., that is, more than five times that of the steel in the reinforced concrete beam.

These comparisons show at a glance that it is cheaper to use concrete than steel to

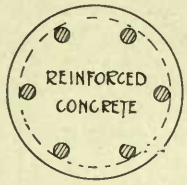


AREA = 5600 SQ. INS.

Fig. 1

STEEL
AREA = 200 SQ. INS.

Fig. 2



AREA OF STEEL = 100 SQ. INS.
TOTAL AREA = 4000 SQ. INS

Fig. 3

Figs. 1 to 3.—Columns of Plain Concrete, Steel, and Reinforced Concrete, respectively: Comparative Diagrams

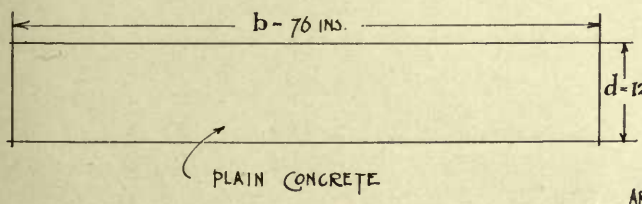


Fig. 4

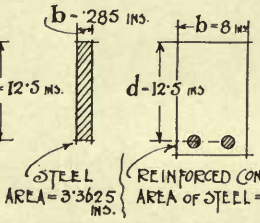


Fig. 5

Fig. 6

Figs. 4 to 6.—Beams of Plain Concrete, Steel, and Reinforced Concrete, respectively: Comparative Diagrams

withstand compression; but that to withstand tensional stress concrete theoretically would cost two and a half times as much as the steel, but, practically, would be impossible. Reinforced concrete effects the compromise between the two; where it costs more than either steel or concrete alone, it offers advantages not possessed by the material with which it is compared. The parallel columns below put the case for the employment of reinforced concrete at a glance.

Concrete	Steel (Mild)	Reinforced Concrete
Weak in tension	Very strong in tension	Strong in tension
Strong in compression	Very strong in compression	Strong in compression
Cannot resist a strong shearing force	Can resist a fairly strong shearing force	Can be made to resist a strong shearing force
A relatively cheap material	A relatively expensive material	Price competitive with that of any other system
Very heavy, strength for strength	Very light, strength for strength	Lighter than plain concrete, strength for strength
Fire-resisting	Destroyed by fire	Fire-resisting
Not attacked by weather and atmosphere; low maintenance cost	Rapidly oxidised; high maintenance cost	Not attacked by weather and atmosphere; low maintenance cost
Easy to place and shape	Difficult to place and shape	Easy to place and shape

Historical Notes

THE history of a practical science is seldom complete and proper without an early reference to the ancient Romans. The history of reinforced concrete is not one of the exceptions. Readers will be familiar with the oft-repeated statement that the Romans commonly used this system in the construction of their public buildings; but while this is an exaggeration of the truth, there are certain unassailable facts that stand out prominently. The Romans made good concrete which can be seen to-day; they anticipated ligno-concrete (wood-reinforced concrete); they combined tiles and concrete; and they roofed the Frigidarium of the Baths of Caracalla with a coarse concrete, made with lime hydraulicised with trass or volcanic scoria, and reinforced with bronze and iron rods. It is fairly certain, however, that there was no general knowledge at the time of the principle of reinforcement as it is understood to-day. There is no reinforcement, be it noted, in the Pont du Gard, a bridge in the south of France erected about 56 B.C., and still in existence—a worthy memorial to honesty of construction; in this bridge the coarse material (stones) and cementing material (lime) were not mixed together, but were apparently placed in alternate layers. It is difficult to believe that for eighteen centuries the idea applied by the old Roman bath builder could have lain dormant; but while we know that concrete of a sort was commonly used, there is no evidence of the principle of reinforcing it, and so extending its usefulness, having been commonly applied at any time during that long period, unless exception is made in the case of the dome of St. Paul's Cathedral (1675–1710), in which, as is well known, Sir Christopher Wren caused chains to be embedded in concrete to help in resisting the lateral thrust. Certain of these chains were examined some years ago, and were found not to have corroded, although the concrete had been continually in a damp state.

Early in the Nineteenth Century.—It is not until 1830 that we find any definite mention of the idea of reinforced concrete, in which year was published J. C. Loudon's "Encyclopædia of Cottage, Farm, and Village Architecture," containing the suggestion that roofs might be constructed of cement in which were embedded iron tie-rods in the form of a lattice work, the whole being cased with flat tiles. Six years previously (in 1824) Joseph Aspdin patented his method of making (portland) cement, and it had already (in 1828) been used extensively in the construction of the Thames Tunnel; but for many years it had to fight opposition and was not in general use. For example, in the year 1840, when, as the result of a strike of carpenters, some fireproof floors were constructed in Paris, they failed to become popular because a local material, gypsum (calcined to form what is now known as "plaster-of-Paris"), was used as the cementing material in the concrete, the result being that the embedded iron soon rusted. These floors were described in a paper read before the Royal Institute of British Architects in 1849 by G. R. Burnell, and more fully described five years later before that body by H. H. Burnell. Two systems were known; in one—the Vaux—round rods, close together, were hooked at each end on to a flat wrought-iron bar lying on its edge; in the other—the Thuasné—iron joists were employed, stirrups hanging from these and containing holes through which the round reinforcing rods passed. In each case the ironwork was embedded in plaster concrete.

About the same time (July 9, 1840) a reinforced ceiling slab was patented by a Frenchman, Louis Leconte; he proposed "the use of trusses of iron plates for floors, from which iron rods were suspended to carry a wire meshwork for sustaining plaster." There is evidence of an indefinite nature that shortly before this period English architects had constructed fireproof floors of concrete or brickwork in which flat iron bars were embedded.

A Concrete Boat.—A fertile period of invention, as relating to the subject under consideration, was the fifties of the nineteenth century. Lambot, a French contractor, had proposed to build the hulls of

boats with concrete, and at the first Paris International Exhibition (1855) he actually showed a flat-shaped boat, with sides 2 in. thick, made of hydraulic lime concrete reinforced with a skeleton of iron rods; and in the same year he patented his invention in Great Britain. At the present time, boats are often built of reinforced concrete.

Wilkinson.—In spite of the systems that had been suggested, and of those that had already been practically tried, the chief credit for the invention of reinforced concrete is commonly ascribed to William Boutland Wilkinson, who, on October 27, 1854, patented a method of constructing a fire-resisting floor of concrete slabs reinforced with a network of flat iron rods placed on edge or with secondhand wire ropes. Wilkinson was a Newcastle-on-Tyne plasterer, and as by 1852 there was established at Gateshead-on-Tyne, by William Aspdin, a cement factory, it may be fairly assumed that Wilkinson was thoroughly familiar with the use of cement concrete. We cannot do better than quote from the admirable "History" printed in the "Lock Woven Mesh Handbook." Wilkinson it was "who first suggested, in his 1854 patent, the use of a layer of sand kept wet upon the surface of a freshly made concrete floor, for the purpose of allowing the concrete to gain the maximum hardness. His patent also included the construction of hollow partition blocks very similar in detail to the interlocking plaster partitions of to-day. The chief object, however, of the patent was the construction of fire-proof floors; these Wilkinson proposed constructing both in arch form and flat, and he suggested reinforcing them with either flat iron bars placed on edge, or with secondhand wire rope. Wilkinson states that the reinforcement was to be placed in the concrete to take the tension, and his drawings clearly show that he thoroughly understood the construction of such floors, for the reinforcement is placed in positions in the arch construction where it could give maximum service, while in floor slabs the rods or wire ropes were bent down at the centre of the span where the maximum bending moment occurs, but in continuous spans, or where built-in, were carried up to the top over the supports, so as to resist the reverse bending moment. The ends of the rope were also directed to be frayed out so as to bond in with the concrete at the ends of the span; the advisability of this was evidently fully appreciated by Wilkinson, namely, to give anchorage or mechanical bond and serve the same purpose as the fishtail or bend at the end of a rod in other systems of a later date. He also described and illustrated independent beams of reinforced concrete. His patent showed a grasp of most of the principles of modern reinforced concrete construction, and it is obvious that it was drawn up from thorough practical experience and perhaps some theoretical grasp of the subject."

In course of time, Wilkinson erected a number of reinforced concrete buildings in Newcastle and elsewhere in the North of England, of which, however, no records are now known to exist.

François Coignet.—Less of the builder and more of the engineer was the French contractor, François Coignet, who in 1855 patented, both in France and England, a system of forming floors by laying iron "planks," or rods, crossing each other, from wall to wall of a building; iron beams could be placed to support the "planks" or rods. A false flooring is supported underneath the reinforcement and the concrete applied. Coignet had at an earlier date invented "Beton Coignet" (concrete for which the hydraulic lime and aggregates were mechanically mixed in certain proportions), and the advantage of reinforcement possibly occurred to him of itself, or he may have remembered the reinforced plaster-concrete experimented with some years before. In the years following the grant of his patent he carried out a variety of works, including a lighthouse at Port Said (Egypt), retaining walls in Paris, and thirty-three miles of aqueduct for the Paris water supply. He undoubtedly commonly applied his system of reinforcement in the construction of these works, but very little is known as to the details.

Dennett, Allen, Ransome, and Scott.—In 1857 (patent dated March 9), a Nottingham builder, C. C. Dennett, took advantage of the introduction of rolled iron joists and constructed a floor with reinforced concrete arches resting on ⊥-beams. The arch reinforcement was of wood or iron. This floor was the prototype of the modern fireproof floor of steel joists and concrete.

The 'sixties also saw a number of notable introductions. In 1862 (January 30) Matthew Allen, a London builder, patented a system of building staircases, floors, etc., with which he afterwards did much business;

iron bars 3 in. by ½ in., on edge, were embedded in concrete (cement 1, cinders, etc., 3) 2 ft. apart near the under-side of the slab, the bars crossing to form a network. The method was sufficiently practicable to be adopted in the construction of the Columbia Market, Shoreditch. Frederick Ransome, in 1865, suggested the construction of girders, etc., by moulding a kind of cement around a hoop-iron skeleton.

The year 1867 saw two important patents. One, by an Englishman, H. Y. D. Scott, related to a floor of concrete with interlaced rods and hoop-iron or wire, supported, not on joists, but on wrought-iron tie-rods embedded in the concrete, and is of interest because the specification alludes to the use of "the tie-rods and hoop-iron taking the tensile strain, and the concrete the compressible," conclusive evidence that the basic principle of reinforced concrete was at that time understood.

Joseph Monier.—The other, a French patent dated 1867, by a Frenchman, Joseph Monier, was for the construction of plant tubs and the like by means of concrete reinforced with a meshwork of rods or wires. Monier started as a gardener, and developed into a manufacturer of gardening tools and appliances. He thought of substituting for the ordinary plant tub a vessel of concrete, but, finding this was brittle, he hit upon the idea of making a skeleton of iron network or trellis and then enveloping it with mortar or concrete. The second Paris International Exhibition (1867) contained examples of construction in the Monier and the Coignet styles. Monier followed up his 1867 patent with further inventions, patented in France in 1873, and, after the Antwerp Exhibition of 1879, at which he exhibited, he sold his inventions to G. A. Wayss (of the firm of Wayss and Co., Germany), who was soon instrumental in introducing the new system of building into a number of European countries. So closely was reinforced concrete identified with Monier on the Continent outside France that it was commonly known as the "Monier system," a term replaced in course of time, and as other systems appeared, by the German-French "eisenbetonbau" and "betoneisenbau" (*eisen* [G.], iron; *béton* [F.], concrete; and *bau* [G.], building or construction—that is, "ferro-concrete construction").

A patent granted to T. Lythgoe and H. Thornton, in February, 1868, is of some interest. The inventors illustrate a floor consisting of ⊥-shaped iron bars suitably spaced and having a concrete filling between them; a hoop-iron reinforcement laces the bars together, and passes alternately over and under them; the bars and the reinforcement are wholly embedded in the concrete.

It cannot help striking the student that the principle and almost the details of reinforcing were re-invented quite a number of times in the middle of the nineteenth century. Under the present laws—British, at any rate—it is doubtful whether any system already mentioned as being invented subsequent to 1840—when, as has been shown, reinforced floors were constructed in Paris—could have been granted a valid patent. The Monier floor, introduced in 1873, is much the same as Allen's, patented in 1862; and Allen's was almost certainly anticipated by Wilkinson's, patented in 1854.

Phillip Brannon.—Brannon's provisional patent in 1870, completed the next year (October 12, 1871), shows that the inventor had grasped the principle of reinforcing concrete. The specification of 1871 shows a concrete floor reinforced with an iron meshwork, as well as the application of reinforced concrete to the construction of sea defences. Brannon was the first to suggest a reinforced concrete pile, the longitudinal reinforcements being of angle-irons united by bars riveted latticewise across them, the whole being wound spirally with wire. Brannon's patents were worked by a company formed by him and known as "The Monolithic Fireproof and Sanitary Construction Works, Ltd.," which in the ensuing years erected several reinforced concrete buildings, some at Walton-on-the-Naze, Essex, and two (houses) in Islington.

Thaddeus Hyatt.—Between the years 1870 and 1877 much interest in the "new" system of construction was exhibited by Thaddeus Hyatt, an American who had become known from his invention of glass pavement lights. It is doubtful whether the published accounts of Monier's and Coignet's work in Europe had attracted his attention. Between 1870 and 1877 he had a series of beams tested by Kirkaldy, and in these beams both single and double reinforcements were tried. He indulged in an interesting series of experiments, and these led him to believe in the necessity of anchoring the ends of the reinforcing rods, for which purpose he fitted them with nuts and washers

to prevent their pulling through the concrete. Between the years 1873 and 1881 he obtained between thirty and forty different patents, most relating to reinforced concrete; and although he did much valuable work in assisting engineers to a proper understanding of the theory of the reinforced concrete beam, it does not appear that commercially Hyatt made any success with concrete construction. Hyatt's book showed that he regarded a reinforced concrete beam as corresponding to a rolled steel joist, the steel rods being considered as equivalent to the bottom flange and the concrete as the top flange, the neutral axis being assumed to lie half-way up the beam. Later, a number of engineers made a similar mistake.

From the 'Seventies to the 'Nineties.—About the year 1870, a number of reinforced concrete sewer pipes were laid in Germany, and these are still in service. In 1877 an experiment was tried at Croydon by W. H. Lascelles, who applied a system based on patents obtained earlier, and built a number of cottages having a timber framework, the slabs being of concrete reinforced with diagonal rods.

We now arrive at a period in which interest was being taken in reinforced concrete construction by engineers in all the progressive countries of Europe. Angelo Lanzoni, of the firm of Lanzoni, Galli and Co., made some applications of the system in Italy in 1878, the first Italian patent being taken out by him five years later, anticipating Monier's Italian patent by four months. In Switzerland, Monier's system was applied to the erection of small vaultings as far back as 1880. In the same year, Rudolph Schüster came to terms with Monier, and in the succeeding years carried out a series of works in Austria, these including large numbers of reservoirs and vats, vaulted flooring, fire-resisting doors, etc. Deep interest had been aroused in Great Britain, but the Monier system, for which a British patent was taken out, dated July 7, 1883, failed to become successful there. The activity was not confined to Europe, for in the United States—where, in 1875, W. E. Wood had built a reinforced concrete house—E. L. Ransome was building reinforced concrete warehouses about 1884, following later with a factory building, the Californian Academy of Science (architect, G. W. Percy), about 1888, and the Museum Building of Leland Stanford, Junior, University (architect, G. W. Percy), in 1892, the last-named building containing spans of 45 ft. and being reinforced throughout. It is stated that the Academy building withstood remarkably well the San Francisco earthquake of 1906. In 1884 Ernest L. Ransome patented in the United States a twisted square bar reinforcement, and in the preceding year John F. Golding obtained an American patent for "slashed metallic screening," otherwise expanded metal, which was employed as a lathing for plaster, its use as a reinforcement for concrete dating from about 1890. Benjamin Scarles took out American and British patents as far back as 1884 for the use of wire cloth as lathing, but even he had been anticipated.

In the year 1885 W. H. Lindsay patented a system of reinforcement by which steel rods were passed through holes made in the webs of I-joists, one at the top and one at the bottom; the vertical pairs crossed, and at the point of intersection they were made to form a loop through which was threaded a rod parallel with the iron joists. About the same time, too, a practical experiment was tried in Lincoln's Inn Fields, London, a block of offices with plain concrete walls and reinforced concrete floors being erected to the designs of William Simmons.

From this point onward, we must omit the names of many patentees and inventors, as during the next twenty years or so there were hundreds of inventions relating to reinforced concrete, and scores of different systems were exploited in Europe and the United States.

In Hungary reinforced concrete was first applied in the erection of stables at the Artillery Barracks, Kassa, in 1886, by Robert Wünsch, of Budapest. To take up the thrust of the vault, the separate metallic ribs were anchored, and the resulting system was at a later date widely applied in Austria and Hungary. Into Hungary the Monier system was introduced in 1887 by Wayss, being first adopted for a series of barrel-arched bridges. Merely mentioning by the way that on March 27, 1890, J. Mayoh patented in Great Britain the use of corrugated plates on edge for strengthening concrete floors; that on July 7, 1891, C. A. Day patented in Great Britain the use of wire lattice suspended over steel joists for reinforcing concrete slabs; and that in the same year Franz P. Meyenberg patented in the United States a floor constructed of

hollow terra-cotta tubes supported on steel rods embedded in concrete above and below, there being loose stirrups hooked round the bottom rods, we may pass to the period in which, in spite of all that had been done before it, reinforced concrete first compelled the serious attention of all progressive engineers.

Edmond Coignet and François Hennebique.—The two great names are those of Edmond Coignet (son of François Coignet) and François Hennebique, the former of whom, by applying the known principles of mechanics, evolved a system of calculation that has proved remarkably truthful, and the latter of whom, basing his methods of calculation upon results obtained in practice, has also made extremely important contributions to the technical consideration of the subject. Coignet as the scientific investigator, and Hennebique as commercial organiser, are properly regarded as "the pioneers of the modern evolution in the art of building." The story has often been told of the opposition which Coignet had to fight in getting the masonry of the proposed new system of main drainage in Paris in 1892 replaced by reinforced concrete. He promised a large saving of money and of time required for construction, and his system, which was finally adopted, was carried out with complete success. Hennebique, having organised a technical staff and licensed a large number of the most influential contractors to work his system, was able to secure between the years 1892 and 1899 work to the total value of two million sterling, representing three thousand constructions, among the most remarkable of these being the bridge of Chatellerault, 460 ft. long, comprising three arches, two of 133 ft. span and one of 167 ft.

Hennebique's first patent dates from 1892 (British patent, No. 14,530), and in this he demonstrates the utility of stirrups to reinforce beams against shear, in which matters he had to an extent been anticipated by Hyatt in 1877 and Meyenberg in 1891. In 1897 Hennebique introduced cranked-up rods, and placed these one above the other, so as to reduce the width of the beam, following (to some extent) the lines laid down by Hyatt in 1877 and F. G. Edwards in 1892, in which latter year M. Koenen and G. A. Wayss, of Germany, patented in England a method of floor construction with rods cranked up at the point of contraflexure, "the parts in tension being strengthened by roughened or serrated metal rods or strips embedded in the structure."

Progress since 1890.—Speaking of the period following the years 1890 to 1892, N. de Tedesco, Ingénieur des Arts et Manufactures, and editor of *Le Ciment*, has remarked that the modern method of building remained for a long time in very few hands, and nothing was to be found in literature concerning the calculation of structures constituted by two different materials. Contractors were obliged to experiment in order to ascertain the most suitable proportions and arrangements of these materials for resisting the determinate load on a determinate span, but they were scarcely able to undertake with advantage investigations of that kind. Such work was rather the rôle of "ingénieurs des ponts et chaussées," but many of these attempted to co-ordinate the results of the tests carried out by Coignet and Hennebique, only to find that it was impossible to deduce mechanical laws from them, inasmuch as the mechanical properties of concrete are variable with the quality of sand or gravel, the cement used, the degree of fluidity of the mixture, the climatic influences during the hardening process, the age, and with many other surrounding conditions. Considère came to the rescue, and by a series of tests undertaken upon small specimens manufactured under uniform conditions he was able to discover the laws governing the deformation of concrete submitted to stress. In 1890 Paul Neumann, in a published memoir, described his mode of calculation, which recognised the relation between the coefficients of elasticity of the two materials, steel and concrete; and about the same time T. Melan published a treatise in which he came to practically the same conclusion as Neumann—namely, that the elastic behaviour of concrete was not the same under compression as it was under tension. Melan, by the way, invented in 1890 the system of constructing floors and vaulting named after him. The first important contribution to a rational theory was the memoir written by Edmond Coignet and N. de Tedesco, and published in the *Bulletin* of the Société des Ingénieurs Civils of France for March, 1894. Into Holland and Denmark, reinforced concrete was introduced about 1890, the floors of the National Gallery at Copenhagen being built of reinforced

concrete in 1891, the same system being used in the next year or so for harbour works, and in 1894 for a 19-metre span bridge for foot passengers, just outside Copenhagen. In Switzerland, where reinforced concrete had previously been used for vaulting, 1892 saw the introduction of Hennebique beams subjected to bending stress. Prof. F. Schüle, of Zurich, has pointed out that two conditions favour the development of the new mode of construction in Switzerland: first, Switzerland has to import all its steel, and reinforced concrete involves a saving in this material not to be undervalued; and, secondly, the excellent position occupied by Switzerland with regard to the manufacture of portland cement. Both in Hungary and Italy, reinforced concrete was now making rapid progress, and by 1896 a bridge having a span of more than 25 metres was being erected at Sarajevo, in Bosnia. In 1897 the Hennebique system was introduced into Great Britain by (the late) L. G. Mouchel, François Hennebique's partner, and since then its progress in this country has been truly remarkable.

Coming to the Paris International Exhibition of 1900, the graceful palaces built by Edmond Coignet and François Hennebique attracted the favourable notice of architects and engineers the wide world over, and thousands of illustrated papers throughout the Continents published photographs of the Château d'Eau, erected by Coignet. If there was anything necessary to stimulate the adoption of reinforced concrete design by the engineers and architects of the day, it was afforded by this advertisement, from which dates an interest in the subject out of all proportion to that formerly existing.

Of the few further inventions which it is worth specially to note may be mentioned the Siegwart beam, introduced in Switzerland in the year 1902. The inventor, a Lucerne architect, desired to avoid construction on the building site other than the actual laying of the prepared beam, and his solution of the problem is satisfactory where the span does not exceed about 5 metres. The Visintini system, invented in Austria and patented in Great Britain in 1902, introduces framed beams into the reinforced concrete construction, and it came into practical employment in 1903, later coming into use for the vaulting of the Evangelical Church at Aussig.

A specification of particular interest is No. 24,371 of 1904, the first British patent granted to Edmond Coignet. It describes and illustrates beams, floors, domes, roofs, columns, and walls, and has a high educational value. The drawings of beams show the tension bars in the lower part of the beam to be connected to the compression bars in the upper part by means of ties. These ties also hook over the tension bars, which extend through the floor slabs and which are bent up to pass over the compression rods.

Notable American Inventions.—As a result of the rare commercial ability behind them, certain American systems have become extremely well known in Great Britain, and perhaps in consequence of this fact there is a common tendency to look upon the United States as the real "home" of reinforced concrete. While there is no doubt that the temperament of the Americans led them to welcome the "new" method of building (as, indeed, they welcome any promising invention), as a matter of fact, there is not, prior to 1892, much that is of truly American origin and of real note in the United States records with regard to reinforced concrete excepting the Thaddeus Hyatt inventions and the Ransome twisted bar, to both of which reference has already been made. In 1892, August C. Storck, of St. Louis, patented a method of strengthening a concrete block with an embedded "brace" of slashed (expanded) metal; and, in the following year, Thomas A. Lee, of Kansas City, filed a specification for a "non-shearing rod for strengthening concrete," two or more rods to be laid together spirally; and E. L. Ransome took out what is apparently his first patent for a definite method of reinforcing concrete, his specification describing and illustrating a slab supported at its ends by walls and reinforced by two bars, a shorter one above a longer one, the bars tapering to nothing at each end.

Some years previous to this, Ransome had erected some important buildings in reinforced concrete.

In 1896, Edwin Thacher, of Detroit, filed an interesting specification showing a concrete arch between abutments, the arch being reinforced with two series of flat metal bars in pairs, one bar of each pair above the other. At intervals, the bars had pegs which projected on each side, apparently to aid adhesion. Ira A. Shaler, of New York, patented in 1900 a method of reinforcing

slabs by means of a metal skeleton comprising longitudinal and cross members, the one welded to the other. Then came, in 1901, two important E. L. Ransome patents; in the earlier, webs of hardened reinforced concrete are placed in position, the spaces between them spanned with falsework, and the top compression member of the floor then made with concrete; in the later, a reinforced concrete floor extends to the exterior face of a building, and there becomes a belt course which caps the piers, a downward extension forming heads or lintels to the windows below. From this date the inventions become too numerous even to mention, but space must be found for a reference to the first Kahn patent, filed on December 11, 1902, and bearing date of August 18, 1903. It describes the now well-known Kahn bar, which has members projecting obliquely so as to form the diagonal members of a part of a truss. An improvement followed in a patent filed on May 4, 1903, and bearing date November 3, 1903. These were the forerunners of a large number of patents by the same inventor, the assignees of the patents being the Trussed Concrete Steel Company, now well known both in the United States and in England.

Concrete: Materials, Proportions and Mixing

What Concrete is.—Concrete is an artificial stone made by cementing together fragments of hard material. The hard material, such as brick, stone, etc., is known as the "aggregate" (literally, something that has been gathered together); the cementing material, such as lime or cement, is termed the "matrix" (literally, but, in the case of concrete, not actually, a mould in which something is embedded). The active agents in the process of solidifying are the cementing material and the water; the aggregate is inactive.

Making a Strong Concrete.—The strength and quality of concrete depend chiefly upon (1) the nature of the aggregate; (2) the nature of the matrix; (3) the proportions in which the materials (including water) are mixed together; and (4) the efficiency of the mixing process.

The compressive strength of concrete is limited by that of the aggregate, and the tensile strength cannot exceed that of the cement bond. The ideal concrete can be pictured as a composite block of stone, brick, etc., in which all the pieces fit together perfectly, being attached one to the other by a film of cement. Such concrete is not attainable in practice, the shapes of the fragments preventing such extremely close fitting, and thus causing the formation of pockets, which become filled with a mixture of cement and sand. Better concrete than is generally made would result from the use of graded aggregates; first, a very coarse aggregate would be selected, then a much finer aggregate to occupy the larger of the voids, then a still finer one to occupy the lesser voids in the aggregate, and finally the cement. According to much of the present practice, there are but two grades in the aggregate, the coarse (stones, gravel, etc.) and the fine (sand).

A little thought will show that the strength of concrete may be affected unfavourably by any one of a number of conditions: Any part of the surface of the aggregate not covered with cement; any grain of sand without a cement coat; any coarse particle of cement which may in the future undergo chemical change; any defective particle of cement that was incapable of doing its duty when brought into contact with moisture in course of mixing; any very fine dust from the aggregate, inert and useless; any crack or fissure in the broken stone; any hollow place resulting from insufficient tamping, etc. Any one of these will constitute a source of weakness.

Concrete-making materials will now be considered in detail. In preparing the information here presented on aggregates, care has been taken to make it conform to a schedule issued with the "Interim Report" of the Special Commission on Concrete Aggregates appointed by the British Fire Prevention Committee.

NATURAL AGGREGATES

Gravel.—This usually consists of smooth, round pebbles with adherent sand, clay, and carbonaceous matter. The large stones need to be broken, and the sand to be sifted. Clayey gravel must be washed, as raw clay seriously affects the strength of the concrete and retards the setting of the cement. It is generally wise, as a matter of fact, to wash all gravel. Soft or shaly should be avoided.

1 cub. ft. of coarse gravel usually weighs about 97 lb., and clean shingle about 93 lb.

Shingle.—This is a kind of gravel found on the seashore, its particles being of a rounded shape. It may contain a small but significant proportion of absorbed free salts, not readily removed by washing. If coarse material, free from sand, is required, it is best to specify, "clean washed and broken shingle."

Sandstone.—This consists of grains of sand, usually quartz, held together by a natural cement, the nature of which may vary in different examples. It is far from being the best of the stone aggregates, owing to its absorbent nature, and its weakness when wetted. It cannot withstand abrasion, and, as a rule, is of doubtful value as a

fire resistant. When it must be used, care should be taken that it is dense, uniform, and as homogeneous as possible in structure. The Interim Report before alluded to states that the weights of sandstones, limestones, quartzites, and rocks of similar character should not be less than 130 lb. per cubic foot, the crushing strength not less than about 3,080 lb. per square inch, and the water absorption not more than 8 per cent. of the weight of the stone after a 24-hour immersion. The aggregate after preparation should be free from all dirt, decomposed rock, clay and organic material.

Limestone.—As in the case of sandstone, the quality varies. The oolites, when of fine grain, answer very well as a concrete aggregate, especially certain crystalline limestones of the magnesian or dolomite series. Limestone consists mainly of calcium carbonate (with or without magnesium carbonate), deposited originally from water, or, in any case, formed through the agency of water. It should have the qualifications mentioned in the "sandstone" paragraph as regards weight and strength. An average weight is about 168 lb. per cubic foot, and the average strength under compression about 4,700 lb. per square inch. Limestones are dangerous in case of fire, becoming converted into lime.

Limestone, sandstone, etc., are sometimes used in a crushed form unscreened so that the very fine pieces and the dust take the place of sand, which can therefore be omitted. There is, however, a strong objection to the use of this material. No flour-like dust, except that of cement, ought to be incorporated into concrete. Stone dust is quite inert, and should it collect, as is likely, in the form of small lumps throughout the mass, it will constitute a source of weakness.

Argillaceous or clayey limestones and the softer and more shaly limestones are undesirable aggregates.

Chalk.—This is a variety of limestone, and varies in hardness, the upper strata being soft, while the middle and lower strata are much harder, and make better concrete than some other materials. Even the top stratum forms a fairly good concrete for walls where the height is inconsiderable. It has been used for two-storey cottages having suspended concrete floors with satisfactory results, being found to resist frost effectually and to possess a considerable crushing stress. The fine sandy portions should be eliminated by screening and coarse, sharp pit or river sand or fine brick sand or shingle substituted. In view of the large chalk area in the south of England, and where better materials are often not available, it might be used to advantage for walls of buildings where economy in construction is desirable.

Quartzite. — This in many respects resembles sandstone, but is harder and denser, resists abrasion better, is not so absorbent, and is therefore a better aggregate. It actually is a sandstone which has become hardened by natural forces and heat.

Flint.—This very hard but brittle stone consists of grey or black siliceous matter, and its probable source was a tiny sponge around which colloidal silica has collected and coagulated from the surrounding water, thus forming the peculiar shapes in which flint occurs. Flints are generally found in bands in one of the chalk series, but they also occur on the seashores, where they have been deposited after the erosion of the chalk beds.

Flints obtained from the surface of arable land are somewhat dirty and require washing, which is more easily performed after they have been broken or crushed. Flint cannot be broken by hand to make a suitable aggregate, but should be crushed by a stone breaker.

Flint weighs about 162 lb. per cubic foot, and the compressive strength is high, about 7,800 lb. per square inch. Flint is liable to fly when subjected to heat, but the tendency is reduced by crushing to small particles less than $\frac{3}{4}$ in. in size. It is not absorbent, and is not durable under shock. Shingle is largely water-worn flint.

Granite.—This is an igneous rock of greater strength under compression (exceeding 16,000 lb. per sq. in.) and of varying specific gravity, an average specimen weighing about 170 lb. per cubic foot.

Granite is an excellent natural aggregate providing that it is free from appreciable quantities of weathered or partially decomposed material. The chief objection to its use is its high specific gravity, which tends to make it settle out of the concrete when the latter is being placed in position, but with a fairly stiff concrete the danger of settlement is small. The granite must be crushed to pieces of suitable size and sifted free from sandy and dust-like particles by screening over a $\frac{1}{4}$-in. mesh. The granite should be well watered immediately before use, as some varieties are highly porous and,

if used dry, greatly reduce the strength of the concrete. If the watering is effected by means of a hose it will incidentally wash off any loosely adherent material, the presence of which is detrimental to the concrete.

The value of granite depends on the sharpness and angularity of the fragments as well as on the roughness of the surface. Those granites should, therefore, be chosen that are naturally brittle, and those which produce rounded fragments in a crusher should be avoided as far as possible. The maximum size of granite particles is usually taken at ¾ in., but for very large blocks fragments up to 2½ in. in diameter are used.

There is little difference in the quality of granites from various sources, so far as their value as aggregates is concerned, so that the chief attention should be paid to selecting a granite of clean appearance, with as rough a surface as possible (to give a good key to the cement), and one of which the fragments are highly angular. Naturally, where a granite can be obtained locally, that will be the granite selected for a particular case.

Other Volcanic Rocks.—These include basalts, lavas, pumice, etc. They should be free from soft portions, and should be tested by immersion in water for 72 hours, at the end of which time there should be no signs of expansion, disintegration, or solution. Basalts, traps, dense lavas, etc., should have a thoroughly vitrified structure, be homogeneous, and show a clean fracture. The basalts make really good aggregate, being strong (compressive strength, about 19,000 lb. per square inch); they are durable under abrasion, and they are of low water absorption; but they have two disadvantages —they are rather heavy (180 lb. per cubic foot), and they have no marked resistance to fire unless care is taken to break them up small. Not so hard as the basalts, traps, etc., are the lavas, including pumice, which, however, for use as aggregates, should be moderately hard, show a bright, silky fracture and be free from dust and impurities.

ARTIFICIAL AGGREGATES

Burnt Clay.—Under this sub-heading may be included broken bricks, clay ballast, broken terra-cotta, broken pottery, etc. The broken bricks should be such as, if whole, would be suitable for building purposes; all soft and under-burned portions, should be discarded. Broken bricks should be free from old mortar, and from dust that will pass through a ⅛-in. mesh. Most broken bricks are very absorbent, and for this reason should be watered before mixing with the other ingredients, as otherwise the cement is robbed of moisture and the setting is injuriously affected. Burnt clay ballast always needs to be inspected carefully for unburnt particles, and it is best to test a sample by soaking in water for a few days to see whether it disintegrates or not; it needs watering in the same way as the broken brick. Porous terra-cotta and unglazed earthenware make good aggregate, and these also should be watered before use; the material should be clean, hard, and well burnt, but any large flat pieces or pieces of distinctly curved shape should be regarded with suspicion, since they do not work in well with the other pieces, and, by "bridging," may easily form cavities.

Residues.—These include blast-furnace slag, clinker, coke breeze, and cinders, which have been proved valuable for special applications, being light and fire-resisting; but it is of the utmost importance that sulphur, if present in anything more than a mere trace, be eliminated, as otherwise it tends to combine with the cement during the process of setting and seriously affects its strength. Then, too, any dense combustible material present (as coal) is a defect which militates against the successful use of residues.

If materials containing sulphur are exposed to the atmosphere for a time and moved occasionally, the sulphur tends to disperse. The smell of the materials when moved is the simplest evidence of the presence of sulphur.

Blast-furnace Slag.—This should come from pig-iron smelting furnaces, and basic slag should be rejected. If there is any doubt as to the sulphur content, and in cases where the source of the slag is known, the works chemist should be asked for the analysis, which in many cases he should be able to supply without much trouble. Only a mere trace of sulphur is permissible, and the slag should be washed to remove dust and possibly some of the sulphur. Washing will also remove any free lime.

Coke Breeze.—The Interim Report, of which mention has already been made, recommends that coke breeze for use as aggregate shall be entirely coke taken from gas retorts, coke ovens or special furnaces, and be absolutely free from clinker, coal,

and all substances that will not float in water, and from any admixture of material taken from the retort furnace or water pan below it, and from cinder, ash, or other adulterant. On no other aggregate is expert opinion divided to such an extent as on coke breeze. All agree that its lightness makes it particularly adapted to suspended floors, and it has another advantage —nails can be driven into it; but it lacks strength. Its weight is about 35 lb. per cubic foot. Being so very porous, it might be thought highly absorbent, but such is not the case, as many of the cavities formed by the bubbles of gas evolved when the coal is heated are sealed and therefore have no communication with the outside, and it is therefore only the surface (and broken) cavities into which the water can penetrate. Some coke breeze has so high a content of sulphur that its use is a positive source of danger, but where this is not the case, the material is a valuable one. Much depends upon the kind of coal of which the breeze is the residue. Thomas Potter, in a paper read before the Concrete Institute, stated that he had used thousands of tons of coke breeze for floors, and did not remember a failure of any kind, and that he had proved it to be one of the best materials for floors and roofs. This, however, is not the experience of all users of the material, but this may be due to sifted ashes ("builders' breeze") having been substituted for true coke ashes.

Clinker, Cinders, etc.—Clinker used for concrete should be the thoroughly burnt and hard waste product of furnaces, free from dust, shale, or free lime. The quality of clinker, cinders, etc., varies greatly, but no material containing ashes, dust, rubbish, and more than a trace of sulphur is suitable as an aggregate. In a later chapter (p. 261), the results of some fire tests of cinder concrete are given, and their somewhat contradictory nature will be noted.

Broken Concrete.—Old and very hard concrete may be broken up so as to pass a 1½-in. ring, well screened in order to remove all dust, and then used as the aggregate in concrete for foundation work or rough walling; but the material would not be sufficiently reliable for floors or fine work.

On no account should old concrete in which calcium sulphate or plaster-of-paris was the matrix, as was the general practice before portland cement was used, be utilised as an aggregate, because when water is added it expands and ruptures the concrete, no matter how long since it was originally employed. Failures of the kind have occurred where the plaster concrete was forty years old.

SIZE OF AGGREGATE

In no work ought the stones to be greater than 2½ in. or 3 in. across, this size being used for large retaining walls, foundations, etc. For walls, the stones should pass a 2-in. mesh and be retained on a 1-in. sieve; and for floors they should pass a ¾-in. mesh and be retained on a ½-in. one. For the surface of concrete floors the stones should be broken so as to pass a ⅜ in. or ½ in. mesh.

The larger the stones, the stronger will be the concrete, the quantity of cement remaining constant. According to experiments undertaken by Messrs. Fuller and Thompson, concretes with stones 2¼ in., 1 in., and ½ in. in diameter respectively required cement in the relative proportions of 6, 7, and 8 to be of the same strengths. The experiments proved, too, that round, water-worn material, such as gravel, gave a denser mass than broken stone, but that the latter gave the stronger concrete. The size of the aggregate particles greatly affects the proportioning of the ingredients.

WASHING COARSE AGGREGATE

It is nearly always desirable to wash the aggregate, and the work should be done on a wooden platform, inclined somewhat to allow water to run off freely. In preparing fairly large quantities for use, two men should shovel the material backwards and forwards, while a third keeps a constant supply of water playing on the aggregate, this being continued till the water runs off clear. The use of a mechanical washer—some form of tower or tumbling drum—gives better results. For good concrete the importance of washing the aggregate cannot be over-estimated. The water leaves the material in the proper condition for causing the cement to adhere closely to it, and the dampness undoubtedly assists the setting.

SAND

There appears to be a difference of opinion as to the function of sand in concrete. Some regard it as a cheapener to be added to the cement to increase its bulk and form a mortar in which the coarse aggregate is

embedded. The writer does not share that view. The sand is essentially the fine part of the aggregate, and the proportion of it to be used depends not on the amount of cement thought to be necessary, but on the percentage of voids in the coarser aggregate. Then, in turn, the proportion of cement will depend on the percentage of voids in the sand and between the sand and the coarser aggregate, there being in addition enough to coat the whole of the aggregate. But for the trouble, sand of two sizes could be used, the finer occupying the voids in the larger. It is significant that experiments carried out by Messrs. Fuller and Thompson seem to prove that the strength and density of concrete are affected by variation in the size of sand grains, and that an excess of fine or medium sand decreases both strength and density.

Sand must be sharp and gritty, and washed thoroughly free from everything that does not consist of small particles of stone, quartz, or non-plastic material. Dust, clay, earth, vegetable matter, and all other plastic substances, are sources of weakness, and the necessity of eliminating them invests with the greatest importance the washing and screening of pit sand. River sand may not need washing, but is improved by screening. Sea sand should have the salt washed out. Pit sand, when of glacial origin, is unsuitable; so is sand containing more than 10 per cent. of very fine grains.

1 cubic foot of pit sand weighs about 102 lb.; river sand, 106 lb.; coarse gravel, 97 lb.; and clean shingle, 93 lb.

Substitutes for Natural Sand.—Where sand is difficult to obtain, a substitute may be used. Indeed, the crushed stone substitutes, washed perfectly free from dust, give a stronger concrete than do some sands, because the artificial crushing, when properly carried out, produces very sharp and angular particles. In addition to crushed stone, sand substitutes include ground burnt clay (pottery, bricks, etc.), slag sand (made by running molten slag into water whereby it becomes granulated), etc. All these need to be washed or screened to remove flour-like dust, because any material that passes through a sieve of 50 meshes to the linear inch is too fine to be satisfactorily used.

Washing Sand.—The best way to wash sand is in a stream of water, the rate of flow of which is adjusted to remove the particles of clay, silt and rock-flour without carrying off the sand. Some mechanical arrangement for keeping the sand in motion and to prevent the formation of eddies is necessary, or some portions will not be well washed. One of the most effective sand-washers consists of a large inclined pipe about 15 ft. in length, in which revolves a screw conveyor. A hopper is fitted to the lower end of the pipe, and through this the dirty sand is supplied, together with the necessary water. The water flows out at the upper end of the pipe carrying the clay, silt and rock-flour with it, and leaving the washed sand behind. As soon as the water flows away in a clear stream the water supply is cut off and the sand removed. This arrangement has been made continuous in action, and greatly improved by the addition of a second pipe about half-way along the larger one; this second pipe carries off the washed sand, and delivers it in a very wet state into wagons or on to a receiving bed. Such an arrangement has been proved to use much less water in proportion to sand than the ordinary cylindrical washers generally employed.

The use of sieves for washing sand is convenient for quantities of not more than 3 to 5 cwt., but is too slow and costly for larger amounts. The sand to be washed is laid on the sieve so as to cover to a thickness of not more than an inch, and the sieve is then partly immersed in a tub of water and shaken to and fro. The idea is that the finer particles will pass through the sieve and fall into the water beneath, but an examination of sand treated in this manner will show that it contains a considerable quantity of fine mud, which is too adhesive and remains suspended too readily in water to be removed by this treatment. Another method of sieve-washing consists in choosing a sieve with a rather coarser mesh than corresponds with the sand, and then washing the sand through the sieve into a tub of water. The coarse material remains on the sieve, and it is supposed that the clay and silt will be removed when the water is poured slowly out of the tub. An examination of the sand left will show, however, that it contains a considerable proportion of adherent clay and rock-flour, and is not really satisfactory for use in concrete.

Where no mechanical means of washing the sand is available, a tank about 6 ft. by 6 ft. by 18 in. should be half-filled with sand and a stream of water turned in. The sand

is stirred continuously with a wooden pole, the stream of water being continued and allowed to overflow until it runs off perfectly clear. The remaining water is then run off, and the sand in the washer removed.

Circular wash-mills, such as those used in the manufacture of cement, are not satisfactory for sand, the best sand washer being, as already mentioned, a stream of water flowing directly forward at a carefully regulated rate, in combination with some mechanical means to keep the sand in vigorous movement whilst it is immersed in the water.

PORTLAND CEMENT

From the times of the Romans until Smeaton's experiments in 1750 there was no material addition to our knowledge of mortars and cements. This intrepid experimenter, urged by the necessity of finding a reliable mortar for use in the construction of the celebrated Eddystone lighthouse, upset all traditions by formulating the theory that the hydraulic or setting property of a lime-mortar did not depend on the hardness or whiteness of the stone from which the lime was made, but on the presence of a quantity of clayey or argillaceous matter interstratified with the stone. Aspdin in 1824 patented portland cement, which was so-called from a fancied resemblance to the well-known Portland building stone. Chiefly owing to a lack of scientific knowledge amongst the earlier makers of portland cement, the first years of its history were full of failures, which tended to bring discredit on the industry and all concerned with it.

Principles of Cement Manufacture.—An endeavour will now be made to trace briefly the various stages in the experiments that led to the discovery of the principles underlying the manufacture of portland cement. The practice of burning limestone for the production of lime to be used as a binding agent or mortar is of great antiquity. The properties that distinguish the burnt lime from its original form of linestone are well known. Briefly, what occurs is that the calcium carbonate ($CaCO_3$) in limestone, when brought to a sufficiently high temperature, loses carbon dioxide (CO_2), and true lime (CaO) remains. Perfectly pure lime when treated with water forms calcium hydrate ($Ca(OH)_2$) or slaked lime, much heat is evolved, and the lime falls to powder. As a rule, the purer the lime the more energetic is the reaction, and conversely the more impure it is the less intense does the action become and the less is the inclination to fall to powder evinced. The chief of the impurities in a poor or slow slaking lime are sand or silica (SiO_2) and alumina (Al_2O_3); when combined these two substances form the basis of all clays or claylike materials.

Smeaton first pointed out that the presence of these clayey materials renders lime hydraulic (gives it the property of setting). James Parker was the first to use naturally occurring nodules of clayey matter which were sufficiently rich in lime to render them hydraulic when burnt and ground, and Aspdin was the first to use fairly pure limestone artificially mixed with clay.

The table shows (roughly) the relative proportions of the principal constituents that are present in (*a*) a pure or "fat" lime, (*b*) a lean, partly hydraulic lime, (*c*) a natural cement, and (*d*) a modern portland cement:—

	(*a*)	(*b*)	(*c*)	(*d*)
Silica . .	1	5	25	22
Alumina . .	·5	5	5	7
Lime . .	98	85	50	65

After burning, these constituents are no longer merely mixed together, but are chemically combined. That is to say, the lime does not exist as such, but with the silica and alumina it forms compounds known as silicates and aluminates and aluminosilicates, just as lime when slaked with water forms a hydrate; thus, without chemical analysis, it is impossible to obtain as such either the lime or the silica and alumina.

Portland cement may, therefore, be regarded as containing compounds of lime with silica or alumina or both, its precise composition depending on many factors and varying slightly with each sample tested.

Composition of Portland Cement.—In order to follow more clearly the various steps in the manufacture of cement, it is desirable that the principles underlying the various processes should be understood. Portland cement is a combination of three bodies, and in order to bring about such a combination the interacting substances must be in such a condition that absolutely intimate union between the various particles may take place. When the reacting substances are liquids, or solids that are soluble in a convenient medium, it is a comparatively easy matter to bring about the desired

union (and this also is the case when union by fusion is employed); but although there is no reason why cement should not be made by mixing substances in solution (a method much used in the production of dental cements), it has been customary hitherto to heat the mixed materials to a state of incipient fusion or vitrification in the commercial manufacture of portland cement.

Hence, to produce a satisfactory portland cement, the raw material must be brought to the finest possible state; also, for physical and chemical reasons, the finished article must consist mainly of flour or inconceivably minute particles. Both the initial and final operations in portland cement making, therefore, entail the employment of methods and machinery that are calculated to bring about a state of intimate union of the materials and uniform fineness of the product.

The intermediate stage, in which the mixture is subjected to a temperature sufficiently high or prolonged to bring about the desired combinations, is where the greatest alterations and improvements of recent years have been effected. In earlier days the kilns, after being loaded by hand, required a week or more of firing, and then needed time to cool; now, the mechanically fed and fired rotary kiln, electrically driven and controlled, turns out in a day more cement clinker than its almost obsolete forerunners produced in a week. For the successful production of a uniform and high-class cement, raw materials of as regular and unvarying a composition as possible are essential.

Cement Works in England.—Owing to the readily available deposits of chalk and clay in the Thames and Medway valleys, numerous works were started and still continue in those neighbourhoods, and it was thought at one time that a good cement could not be produced save from these materials; but whilst London cement still holds its own, the growth of knowledge and extended experience has shown that in many parts of Great and Greater Britain other materials exist from which some of the best cement may be manufactured, particularly in the Tyne Valley, in Warwickshire and Cambridgeshire. Probably, as landowners avail themselves of the knowledge and experience now obtainable, an increasingly large number of factories will spring up in the neighbourhood of the large centres of population where suitable raw materials and fuel may be found, together with a steady demand for a sound cement.

Preparing Materials for the Kilns.—Two chief methods are employed in preparing the materials for the kilns. They are known as the "wet" and "dry" processes respectively. The former is used chiefly for chalk, clay, and soft materials, and the latter for argillaceous limestones and harder rocks. In some works, a modification or combination of both methods is employed.

The Wet Process.—This is used for most of the cement made in England at the present time, and is chiefly applied to the vast deposits of chalk found in Kent and Essex, in which districts the chalk is obtained by quarrying, and the men employed in this work are known as "chalkies." The material is sometimes loosened by blasting, but this is a practice not often necessary. After removal of the bull-head or gravelly soil, the chalk is loosened by crowbar or pick and slides down the face of the cliff, and is directed by temporary wooden shoots into the wagons awaiting its descent. Large quantities of flints occur in layers in the chalk deposits, and these are removed either at the quarry or when the wagons are unloaded at the washmill. The chalk wagons are conveyed to their destination by small locomotives. These locomotives also bring the clay in trucks from the pit, or, more generally, from the wharf whither it has been brought by barges from the dredging grounds on the Medway. In one of the most up-to-date works in Great Britain the clay is now dumped into the washmills at the pit. The object of the washmill is to separate the coarse impurities and to mix the fine ones together intimately.

The washmill consists essentially of one or more circular tanks, usually about 15 ft. in diameter and 5 ft. deep. In the centre is a brickwork pier carrying a vertical shaft bearing radial arms from which is suspended a series of harrows. The central shaft is rotated by suitable gearing, and the steel teeth of the harrows run in the annular space between the central pier and the mill walls. Into this mill the chalk and clay are introduced in the proper proportions, together with a sufficient amount of water to form with the disintegrated chalk and clay a liquid of creamy consistency known as slurry. Often the chalk and clay are treated separately in washmills, and the mixed

slurries are then treated in another mill to secure intimate contact. As the slurry leaves the mill it passes through screens and fine sieves, so that any coarse particles are retained in the mill. From the washmills the slurry is led through the grinding mills, which formerly consisted of French burr stones, but these are being gradually replaced by tube mills and more effective grinding machines.

Both the materials led into the mills and the resulting slurry should be under the immediate control of the works chemist or his assistants. The rule-of-thumb methods of former times are now superseded by very careful sampling and speedy analysis which permit the composition of the mixture to be accurately adjusted, if necessary. As a general rule, the amount of chalk is about three times that of clay; in some cases the mater als are weighed, and in other cases measured. In either case, the chemist should be able readily to alter the relative proportions of the ingredients, and this is generally accomplished by varying the amount of chalk, the clay remaining constant in weight or volume. As the slurry passes through the mill samples are taken at short, regular periods; these samples are carefully mixed, and an average sample is then sent to the laboratory for analysis. This analysis consists generally of a speedy and reasonably accurate estimation of the calcium carbonate present, together with a test of the fineness and moisture in order to ascertain whether the grinding apparatus is working satisfactorily.

Formerly the slurry was pumped from the washmills into huge settling tanks or backs, where it was allowed to dry, and it was then dug out and loaded into upright bottle kilns. After the slurry had been pumped into the backs there was little possibility of altering its composition, and so, right or wrong, it went into the kilns. At a later date, the slurry was pumped from the washmills into chambers or tunnels through or over which passed the waste heat from the kilns, and this was a decided economy, saving both time and labour. But this practice is now giving way to the method of burning in rotary kilns with previous mixing in tanks, the slurry being constantly mechanically stirred in order to obtain a material of more uniform composition. This method is also of great advantage to the chemist, as it affords another opportunity of testing the composition of the mixture and correcting it if necessary.

The Dry Process.—As already mentioned, the dry process is often used for limestones, shales, etc., in the manufacture of portland cement, which cannot be reduced to powder or slurry merely by stirring them with water in a washmill. The chief materials for which it is used are the limestones and shales of the lias formations in Warwickshire, and the limestones and harder clays of Wales. The materials are dried and reduced to a fine powder by means of crushers similar to those used for grinding clinker. The powder is mixed with water into a stiff paste in a pugmill or mixer, consisting of an open trough fitted with revolving blades on a horizontal shaft. The paste is broken into irregular lumps or is compressed into bricks, which are dried and then taken to the kilns. If a tube mill is used for crushing the material, an open pan mixer may be used. Where a rotary kiln is employed, there is no need to dry the lumps or bricks of "compo," and some firms prefer to use a very soft paste and to load this direct into the rotary kiln.

Opinions are divided as to the relative values of the cement produced by the wet and dry processes respectively. The choice of one or other method depends chiefly on the nature of the raw materials, so that a detailed comparison is of little value, and concerns the materials rather than the method employed. For this reason such comparisons are best avoided as being often misleading and based on insufficient grounds. It does, however, appear to be a fact that the clinker produced from materials treated by the dry method is somewhat denser and more difficult to grind than that obtained when the wet process is used.

Producing the Cement Clinker.—The mixed materials, in the form of liquid slurry or of solid lumps or bricks, must be heated sufficiently to cause it to fuse partially or to vitrify, in order to effect the necessary chemical combinations and to produce cement clinker. Until recently, this heating was effected in kilns of two types—(*a*) vertical shaft or bottle kilns, and (*b*) continuous Hoffmann kilns. The former consist of an upright shaft into which fuel and lumps of cement mixture are fed so as to form alternating layers, in a manner resembling the burning of lime. When the kiln has been filled the fuel is lighted, and when it has all

been burned the clinker is withdrawn from the bottom of the shaft. This intermittent working produces a cement of variable quality, but, with care, satisfactory results are obtained. The Hoffmann kiln consists of a ring-shaped space in which the lumps of cement mixture are stacked, and the fuel is burned in hollow pillars, about 6 ft. apart, left in filling the kiln. Its action is quite continuous, one part of the kiln being filled and another emptied whilst others are being burned. Owing to the regenerative use of the air and fuel gases, this kiln is highly economical in fuel, but the labour of stacking the lumps of material and of withdrawing the clinker by hand makes it more expensive than the rotary kiln, which is rapidly replacing all other types, one of its advantages being that it can burn liquid slurry.

Although rotary kilns were introduced at the beginning of the eighties of the nineteenth century, their extensive and successful use is a matter of comparatively recent history. The rotary kiln as now generally used may be briefly described as a steel or iron cylinder from 5 ft. to 9 ft. in diameter and from 75 ft. to 250 ft. in length; many kilns originally of the former length have been extended, and now 150 ft. is the more general length. This cylinder is lined with a material that is calculated to withstand both the high temperature and also the action of the highly basic, almost molten, clinker. The cylinder is erected at an inclination varying with the materials that are to be used and the intended speed of rotation, the modern tendency being to decrease the angle and increase the speed. The kiln is supported on roller bearings and set in brickwork hoods at each end, several kilns being often connected to one chimney shaft by means of flues. It is not necessary to enter into the details of driving and controlling the machines; electric power is usually employed, and by means of an ingenious device the speed of rotation and the fuel or raw material feed may be varied at will. Very finely powdered coal, which has been passed through a rotary drier before grinding, is the usual fuel; the coal is brought from the grinding mills by screw or other conveyers to hoppers at the lower end of the kilns, and is blown into the latter by steam or air supplied by a fan. The raw mixture, or slurry, is pumped into the upper end of the kilns, down which it is carried by the rotary motion. During its passage through the kiln the moisture is first driven off, then the carbon dioxide is removed, and finally the lime thus produced combines in the hottest part of the kiln with the silica and alumina of the clay; the semi-molten globules that are thus formed drop out at the lower end of the kiln, and then pass through cooling cylinders. Cold air passes up through these cylinders, which in construction are very similar to the kilns; and in this way the clinker is cooled and the now hot air is supplied to the kilns. The clinker thus produced is of a dark greenish colour, in pieces about the size of hazel nuts, and is received into hopper trucks, which, as soon as they are filled, are conveyed to the weighing bridge and thence on to the storage hopper or dry grinding mill.

Grinding the Clinker.—Dry grinding with burr stones has now become quite obsolete, the work being done largely by ball and tube mills, pendulum mills, or other modern contrivances, no preliminary crushing of the rotary clinker being necessary.

The form of pendulum mill chiefly used in England is the Griffin mill, consisting essentially of a horizontal driving pulley, from which the shaft is suspended by a universal bearing. To the lower end of the shaft a crushing roll is attached, which is thus free to swing in any direction within the pan containing the ring against which the roll works, the pulverising being done between the roll and the edge of the pan. Just above the roll is a fan, and on the under side are a number of ploughs. The roll is within the ring, and centrifugal action carries it against the edge of the pan. When the clinker is fed into the mill it is thrown up by the ploughs between the roll and the edge of the pan. When ground sufficiently fine the cement passes out through a screen surrounding the pan, which for that purpose has a number of openings downwards through it. The whole grinding part of the mill is covered with a conical sheet-iron case, and the revolving fan draws in air through the top and forces the cement out through the screens.

The ball mill, briefly described, consists of a cylinder revolving round a horizontal axle. The circumference is formed of perforated steel plates arranged to overlap each other. Outside the plates are coarse screens, beyond which again finer screens may be placed. Sometimes an air-separator replaces the screens. The clinker is fed into the mill

through a hollow trunnion, and in the mill are placed a number of steel balls which are carried round and dropped from plate to plate as the mill revolves, thus grinding the material that is fed into the mill.

As a rule, the ball mill is only used for coarsely grinding the clinker, which is then passed into a tube mill, which consists of an iron shell from 16 ft. to 22 ft. long and about 5 ft. in diameter. This shell is about half-filled with flint pebbles, and the mill is fed and discharged by means of hollow shafts at each end; as the mill rotates the pebbles are carried partly round up the side of the mill until a point is reached where they drop back in a cascade upon the material below, thereby reducing it, in time, to finest possible powder. The material to be ground enters at one end of the tube and gradually works its way to the other end, where it is discharged on to screens or into an air-separator, the coarse material being returned to the mill to be re-ground. The pebbles are fed into the mill through a manhole, and are prevented from leaving by the smallness of the apertures in the mill. The tube mill is only used as a finishing mill, and usually it must be worked in conjunction with a coarse grinding mill, a ball mill being admirable for this purpose.

From the mills the now finely ground cement is carried by belt, screw, or other conveyers to the storehouse, and there packed in sacks or casks or stored in bins until required.

Regulating the Setting of Cement.—Cement made from clinker burnt in shaft or Hoffmann kilns is relatively slow in setting, but that made under the best modern conditions and burnt in rotary kilns sets with inconvenient rapidity and requires the addition of some retarding agent. Several substances are available, the one most generally employed being gypsum (plaster-of-paris) or other forms of calcium sulphate ($CaSO_4$). Most specifications of portland cement recognise the necessity for this retardation of the setting, and they permit the addition of not more than 2 per cent. of one of the substances above mentioned, the addition of the substance being made during the operation of grinding. Another plan now frequently followed is the blowing of steam into the tube mill; in this way any traces of free lime are hydrated. The cement thus manufactured should be fit for use immediately after grinding, which was not the case in former days; it used to be inadvisable to use cement freshly made. On no account should the contractor keep cement long on hand before using.

Testing Cement.—In the early days of the cement industry the methods of testing employed were on all fours with those obtaining in the majority of the cement works of that period, namely, if existent at all, entirely unscientific, rough, and crude. The apparent antipathy of the earlier cement makers towards chemists or other trained scientific workers and methods was only overcome when, as a result of numerous disasters, their aid was perforce invoked. This probably accounts for the many useless tests introduced at a later period of the industry, every engineer, architect, or surveyor, in their anxiety to avoid disaster and disappointment, devising tests that pleased their individual fancy. At this time, also, every works of note had its own system of mechanical and physical tests, according to the ideas of those in control. This diversity of opinion and specifications continued until the opening years of the present century, when, in 1903, a Committee on Cement was appointed by the Engineering Standards Committee, which is supported by the principal engineering institutions and societies.

The sectional committee on cement which drafted and revised what is now the recognised British Standard Specification (obtainable through any bookseller for 5s.), is composed of representatives of various institutions, public bodies, cement manufacturers, experts, and large users. Every buyer or large user of cement is recommended to obtain a copy of this specification, and to use only cement that is guaranteed to comply with its requirements. It will at once be recognised that it is impossible for any inexperienced person to carry out many of the physical and mechanical tests therein described, whilst the question of ascertaining the chemical composition is one for the trained analytical chemist. The users of cement, therefore, must of necessity depend largely upon the guarantee of the manufacturer or employ the service of an experienced tester or expert. If, however, the quantity purchased, or the importance of the work, does not warrant the expense thus involved, the user, having taken the preliminary precaution of obtaining a well-known brand of cement, may for his own

gratification carry out a few simple tests that will at once tell him whether the cement is suitable for the object in view.

Before describing these "unofficial" tests, it may be well if we briefly enumerate the requirements of the British Standard Specification. The following is abstracted, by permission of the Engineering Standards Committee, from Report No. 12 (revised August, 1910), "British Standard Specification for Portland Cement," which specifies the limits of the following properties: (*a*) fineness, (*b*) specific gravity, (*c*) chemical composition, (*d*) tensile strength (neat cement), (*e*) tensile strength (cement and sand), (*f*) setting time, (*g*) soundness.

For tests (*d*) and (*e*) special apparatus is required, whilst (*b*) and (*c*) can only be properly performed in a chemical laboratory or specially fitted testing department. In order properly to determine (*a*), a good balance and set of weights and sieves are required, and for the official method of ascertaining the "setting time" (*f*) a Vicat needle is necessary. A little piece of apparatus must be obtained to carry out the Le Chatelier test for "soundness" (*g*). Each one of these tests requires some amount of experience before reliable and concordant results can be expected.

It will at once be realised that cement testing is not work that can rightly be left to the "odd job" man, but requires care and a fair amount of intelligence and common sense. The tests formulated above are such as any normal, unadulterated, and well-made cement will, as a rule, satisfactorily pass.

At the same time, it is possible that cements not absolutely in accordance with the requirements of this specification may prove satisfactory and reliable in practice, though the employment of cement that is guaranteed to pass the test enumerated is strongly advocated.

Fineness Test for Cement.—The object of determining the degree of fineness of cement is to ascertain that its particles are in such a condition as to practically constitute an impalpable powder. It may be definitely stated that it is only the "flour" in a cement that possesses any real cementitious properties. All the moderately coarse particles are not only useless, but may prove to be a source of real danger. The determination of the fineness is a test that may be easily carried out by any intelligent man. In order to make the test quantitative, a balance and weights, and two "Standard Wire" sieves of 180 by 180, that is 32,400 meshes, and 76 by 76 (5,776) meshes, per square inch are required. One hundred grams (metric system) or 4 oz. of cement is carefully and continuously sifted for fifteen minutes on each sieve. The residue on the 180 sieve must not exceed 18 per cent., that is 18 grams or ·72 oz. (nearly $\frac{3}{4}$ oz.), and 3 per cent. on the "76." The latter constitutes practically an unweighable amount except a delicate balance and weights be used.

Quantitatively, a rough test of the fineness of the cement may be made by sifting a handful of cement through the "76" sieve; there should be practically nothing left on the sieve.

Specific Gravity Test.—This takes the place of the now obsolete "weight per bushel" test. It cannot be carried out except in a properly equipped testing-room. The object of this test is to eliminate adulterated or lightly burnt cement.

The British Standard Specification requires the *minimum* specific gravity of portland cement to be 3·15 at works, and 3·10 after delivery; taking the higher figure, 1 cub. ft. of solid cement would weigh 196 lb. It is not easy to get uniform results in determining the specific gravity of a fine powder, and the specific gravity will vary with the extent to which the cement is packed into the weighing vessel, and also with the conditions of manufacture. The accepted weight of 1 cub. ft. is 90 lb., although actually it may vary between 75 lb. and 110 lb. A cement ton varies slightly. It may be 2,200 lb., made up of 10 two-cental sacks, or it may be 2,244 lb., consisting of either 11 sacks, each of 204 lb., or 12, each of 187 lb. All these weights are net.

Analytical Test.—The services of an experienced analytical chemist should be employed if there is any doubt as to the genuineness of the cement. The addition of ground slag, bricks, or other adulterant can only be proved by accurate analysis. Certain definite ratios between the "basic" and "acid" components of the cement are stated in the Standard Specification, as well as limits for the percentages of magnesia, sulphuric anhydride, and water.

Tensile Strength Test.—The cement, both "neat" and "with sand," is made up into briquettes under certain stated conditions, and the tensile stress of the briquettes

ascertained by pulling each piece until it breaks in a special machine after the briquettes have been stored for stated periods of time. A properly equipped testing-room and the services of an experienced gauger are required.

"Setting Time" Test.—The study of the "setting time" of a sample of cement when gauged with water is, although apparently simple, one of great complexity. In order to obtain concordant results it is necessary to strictly adhere to the conditions expressed in the Specification. A Vicat needle and gauging tools are necessary. The setting time may be roughly ascertained by gauging up a small portion of the cement with sufficient clean water to form a thick paste. A pat is then made up on a small sheet of glass or piece of slate, and the surface tested with the thumb-nail until it is impossible to make any impression thereon. It is possible in this way to distinguish between a "quick" and "slow" setting cement, but not with that accuracy obtainable with the standard needle. Care must be taken in gauging not to "work" the cement after it shows any signs of "setting."

Soundness Tests.—The object of all the suggested tests for "soundness" is to ascertain rapidly the probable effect of time upon the set cement. It is a practical endeavour to prove that the physical state and chemical composition of the cement are as nearly as possible correct. At present there are no fully satisfactory methods of attaining this object. The Le Chatelier test has been adopted by the Engineering Standards Committee because it is the only simple quantitative method that has yet been devised. Many other tests have been suggested, but they are either not quantitative or else require expensive and complicated or delicate apparatus. For a full description of the test, the British Standard Specification should be consulted. Briefly, it consists in filling a small mould, consisting of a split brass ring fitted with indicating needles, with the gauged cement. The mould and contents is then immersed in water for twenty-four hours, removed, and the distance between the indicating needles measured in millimetres. The apparatus is then placed in water again, which is caused to boil continuously for six hours. When cool, the distance between the indicating points is again measured, and any increase noted. A figure is thus obtained which should not exceed the prescribed limits of the Specification. As before mentioned, this is the only simple quantitative test for "soundness" that can be performed without the aid of expensive and delicate apparatus.

It may not be out of place, however, to briefly describe one or two quantitative tests that require practically no apparatus. Some people, before testing for setting time and soundness, spread out the cement to be examined in a layer 3 in. deep for twenty-four hours at a normal temperature; but the practice is not recommended. It should be understood that no reliable results can be expected unless the operator knows how to gauge cement properly and make it up into a pat on a piece of clean glass. It is not a simple matter to do this, and experience is the only teacher.

An excellent test is to plunge a pat, before the cement sets, into cold water. A good cement will easily stand this test, a faulty one will either fall to pieces almost immediately, or show signs of cracking at the end of twelve or twenty-four hours. A quantitative "hot" or accelerated test is to place a pat after setting into cold water, which is then gently caused to boil and kept boiling for six hours. The pat should remain on the glass, and not show any signs of cracking or disintegration. It is not serious if the pat leaves the glass, but is otherwise sound and not buckled or cracked.

It may be remarked that the British Standard Specification does not require a cement to undergo or stand either of the tests last mentioned, but in most cases any cement that complies with the requirements of the Specification will also pass these qualitative tests.

NATURAL CEMENTS

"Natural" cement manufacture differs from that of portland cement, which is always regarded as an "artificial" cement, in one important particular: for natural cement, the material—highly calcareous marl or septarian nodules—is dug, calcined, and ground, thus producing a cement of varying composition and quality; whereas, for portland cement, additions are made to the raw material in accordance with the results of skilled tests, so as to obtain a cement of perfectly uniform composition. The cement is also tested at various points in the manufacturing process, and the resultant product is therefore a reliable and uniform article,

whereas the natural cements vary greatly, even at the same works.

A natural cement very inferior to portland cement is produced in Belgium, and a few years ago large quantities of it were used in Great Britain, the material being introduced into this country by the most unscrupulous methods. The packages were put up in close imitation of those containing genuine portland cement of British origin and often bore labels of well-known British brands. The importation of this fictitious portland cement is now practically dead, but as recently as 1904 it is computed that nearly 234,000 tons entered Great Britain.

SLAG CEMENT

Ordinary blast-furnace slag without any addition does not, as a rule, possess cementitious properties; it requires the addition of lime powder to make it hydraulic. Cement made from blast-furnace slag, and known as "slag cement," has, therefore, chiefly consisted in the past of a mixture of granulated blast-furnace slag and finely powdered lime. In certain patented processes, a cement is made by causing the molten slag to become impregnated with alkaline salts in solution.

A small proportion of granulated blast-furnace slag when finely ground with good cement clinker is stated to make the concrete of which it forms part less pervious to water, and substantially to increase its mechanical strength; burnt clay, trass, or other puzzolanic material, is said to have the same effect. There are, however, reasons for supposing that these statements have been originally made by those interested in the utilisation of slag cements or adulterated portland cements, and that they are subject to the suspicion of being biassed.

Slag cements are now made by a variety of processes, and in some cases they resemble true portland cements so closely in composition and properties as to be indistinguishable from them. Most slag cements, on the contrary, are quite distinct from portland cements, and are nothing more than artificial puzzolanas. Such cements should be rigidly excluded from admixture with portland cements or from sale under this title; their nature should be clearly stated, as puzzolanic cements are very inferior to portland cements. Apart from the careful adjustment of the composition of the slags by the addition of limestone, followed by a recalcination of the mixture in the kiln (whereby a true portland cement is formed), it does not appear probable that there will be any improvement in the manufacture of cement from blast furnace slag that will render it a keen competitor of true portland cement. It must be remembered that slag is a waste product, and that in the manufacture of iron from its various ores the composition of the slag must vary according to the character of the ore smelted, without any consideration relative to its ultimate employment as a cement.

Iron Portland Cement.—For some years a mixture of portland cement and granulated blast furnace slag has been used in Germany under the term "Iron Portland Cement," and has been the subject of much discussion. The portland cement is made by mixing the granulated slag with lime and heating to sintering, and the clinker so produced is mixed with about three-sevenths of its weight of slag. The mixture is then ground and forms the iron portland cement of commerce. A commission appointed by the Prussian Minister of Public Works reported in 1908 that iron portland cement and portland cement may be considered of equal value if in air-hardening tests under standard conditions the iron portland cement gives satisfactory results; but this has not been borne out by the iron portland cements placed upon the British markets, these having a variable composition and often containing deleterious substances.

LIME CONCRETE

Concrete made with lime is weaker and more porous than that made with portland cement. Lime concrete will not always set properly in a damp soil, and it has therefore little value beyond that of a simple hard core. Chalk lime is quite unsuitable for damp situations, only the best hydraulic limes being permissible. The best lime that can be used for concrete is that known as blue lias lime, which is made from a limestone containing approximately 79 per cent. of carbonate of lime and 17 per cent. of clay; and it is the clay content that makes the lime hydraulic, that is, capable of setting under water. The less the proportion of uncombined lime present, the greater is the hydraulicity. Lime concrete is not used for reinforced work where portland cement is obtainable.

WATER FOR CONCRETE MIXING

The water used in mixing the concrete should be clean and preferably fresh. The use of hard water causes a white efflorescence to come out on the walls after a time, similar to the well-known patches that appear on red-brick fronts. On no account should dirty water, or water containing organic matter, as that from stagnant pools, be used. Warm water causes the cement to set a little quicker. An excess or insufficiency of water in mixing is to be guarded against—20 gals. of water per cubic yard is a fair amount, but a larger or smaller quantity may be necessary, according to the nature of the aggregate. Whether the concrete is wet enough can be judged after it is deposited and rammed; there should be just a slight wetness of the top surface. When there is not enough water, the cement does not set properly; where there is too much, the cement may be washed off the aggregate, and, if the forms are roughly made, some of the thin stuff will run away. Another point is that a moderately wet mixture prevents the formation of voids and secures sufficient plasticity to ensure a complete filling of the space round and below the steel reinforcement; but an excessively wet concrete contains numerous globules of water, which, when absorbed, leave the concrete porous, and tend to accumulate on the surface of the reinforcement, particularly on the under side. According to the Proceedings of the American Society of Civil Engineers, the weakening of the bond from this cause was evident in certain beams in which the adhesion was noticeably weak, the water cavities being apparent at the bottom and sides of the steel bars.

Sea-water should be avoided, as it contains many impurities.

PROPORTIONING CONCRETE

Correct practice in proportioning concrete is based upon a proper understanding of the purpose of the ingredients, which has already been explained. The cement, in addition to its general binding power, fills the interstices in the sand, and the sand the interstices in the aggregate. Thus, the coarser the aggregate, the more the sand required; and the coarser the sand, the greater the proportion of the cement that must be used, though the addition of a small amount of a much finer sand is a more economical proceeding. The coarser the cement, the greater the porosity of the resultant concrete; and by filling the fine interstices in the cement itself with a still finer substance (hydraulic lime, or extremely finely ground cement-flour), it is possible to produce a non-porous concrete.

Round pebbles do not interlock with one another so well as do materials of an angular shape, and as a result more sand and cement are necessary to fill the spaces (in concrete parlance termed "voids").

The essential condition is the production of a solid body without voids, all the pieces of aggregate and particles of sand being united together by the cement matrix.

Considered in the light of the above, concrete consists of but two component parts, aggregate and matrix; and the sand may be regarded as the finer part of the aggregate, its purpose being merely to occupy the spaces between the coarser pieces. Thus, if the sand were omitted, cement to nearly the same bulk as that of the sand would have to be added.

It follows, then, that although success in concrete making is often thought to depend entirely upon strict adherence to a formula, there is no one formula of any value for general adoption; and the correct proportions ought always to be settled after experimenting with the particular coarse aggregate and sand which is to be used on the job.

Unfortunately the composition of concrete is frequently settled in a very haphazard manner. It is not uncommon to find specifications in which one part of cement is assigned to so many parts of gravel, sand, and broken stone, without apparently any systematic determination as to whether the sand and cement combined together will entirely fill the interstices in the larger materials. The aggregate should by no means be uniform in size, as already explained. The average percentage of voids in a well-graded aggregate of crushed stone is from 36 per cent. to 48 per cent.; in gravel the percentage is a little less, varying from 30 per cent. to 40 per cent. Sand usually contains 40 per cent. to 50 per cent. of voids—the actual proportion depends on the degree of uniformity in the size of the grains.

From what has been said, the reader will know better than to fall into the error of supposing that, for example, 1 measure of cement, $2\frac{1}{2}$ of sand, and 5 of stone will make $8\frac{1}{2}$ measures of concrete. Mixed dry, the $8\frac{1}{2}$ measures would make theoretically only

5 measures, or slightly more, because the cement and sand should just fill the voids between the stones.

Determining Percentage of Voids in an Aggregate.—A method of determining the percentage of voids in an aggregate is by means of two watertight vessels of known capacity; for the larger vessel, if the concrete is for reinforced work, a convenient capacity would be 1 cubic foot; and for the smaller vessel an imperial pint measure, there being 49·82 pints in 1 cubic foot, which is near enough to take as 50; or the correct capacity could be arrived at by experiment based on the fact that a pint of water weighs 20 oz. Fill the larger vessel with the aggregate, slightly shake down, and strike off level with the top. Should the aggregate be of a porous material, it is first saturated with water and drained before placing in the vessel, as otherwise it will absorb water into itself during the measuring of the voids, thereby giving a greater value for the voids than is actually the case. Now pour water into the vessel containing the aggregate by means of the pint measure, until no more water can be introduced; note the amount required, and take care to do the work accurately and without spilling any of the water. From the number of pints required to fill the vessel in addition to the aggregate, the percentage of the voids is found by the following rule:—

$$\frac{\text{Number of pints}}{50} \times 100, \text{ or } N \times 2,$$

N being the number of pints to fill the vessel containing the aggregate, and 50 (49·82) the number of pints in 1 cubic foot.

The following is a more accurate method, and it involves the use of a galvanised iron bucket and a weighing machine. First weigh the empty bucket denoting weight by W; secondly, weigh the bucket full of water, denoting weight by W_1; thirdly, weigh bucket full of aggregate or sand, denoting weight by W_2; fourthly, weigh the bucket full of aggregate and water, denoting weight by W_3. Then the cubic contents C of the bucket in cubic feet will be $C = \frac{W_1 - W}{62{\cdot}4}$, since a cubic foot of water weighs 62·4 lb. The volume of the voids $V = \frac{W_3 - W_2}{62{\cdot}4}$. The percentage of the voids in the sand or aggregate will be $\frac{V \times 100}{C}$.

With sand, it is difficult to drive out with water all the air contained in the voids, and therefore an error of from 8 per cent. to 10 per cent. may be easily made. This can be avoided by pouring a measured volume of dry sand slowly into a graduated glass vessel containing water; the sand sinks to the bottom free from air bubbles, and the volume of displaced water may be measured and deducted from the volume of the sand, the difference being the voids.

As an illustration of the first method, assume that it requires 24 pints of water to fill the foot cube (50-pint) vessel of aggregate; then the percentage of voids will be $\frac{24}{50} \times 100 = 48$ per cent. By the second method, assuming the bucket to weigh 4·12 lb. = W; when full of water, 41·5 lb. = W_1; full of dry aggregate, 61 lb. = W_2; and when full of aggregate and water, 79 lb. = W_3.

The volume C of the bucket will be $\frac{41{\cdot}5 - 4{\cdot}12}{62{\cdot}4}$ = (approx.) 0·6 cubic foot.

The volume of the voids is $\frac{79 - 61}{62{\cdot}4} =$ 0·29 cubic foot.

The percentage of voids will be

$$\frac{{\cdot}29 \times 100}{{\cdot}6} = 48 \text{ (approx.)}.$$

Next determine the voids in the sand by the same method as for the coarser aggregate. Then proportion the cement and sand so that the cement paste will be 10 per cent. in excess of the voids.

Now, cement when mixed with water reduces very much in volume, and it has been found that it takes approximately 100 lb. of cement to make 1 cubic foot of cement paste. It is a simple matter to provide 100 lb. of cement to each cubic foot of voids in the sand, allowing an extra 10 per cent. in excess of the voids, and to proportion the volume of the aggregate so that all the voids are filled with the cement and sand mortar with an excess of 10 per cent.

One bag of cement (eleven bags to the ton) weighs 204 lb. net, and will make 2 cubic feet of cement paste. This is a convenient basis on which to proportion the volume of sand and aggregate for the voids to be filled; the volume of the cement being thus fixed as one bag, no measuring or weighing of cement will be required, and the resulting concrete will not be too large in volume to handle before the first set begins.

The volume of sand in which the voids will be filled with a given volume of cement paste is found by the following rule, which also answers for finding the volume of aggregate to be filled by a given volume of cement and sand: $\frac{C}{V + (\frac{1}{10} \times V)}$, where C is the volume of cement paste (or cement paste plus sand), and V the percentage of voids in the sand (or aggregate).

Applying this rule to a particular case, assume that the voids in the sand are 38 per cent. and in the aggregate 48 per cent. (That is, taking the bulk of the sand and aggregate as 1 each, the voids are represented by 0·38 and 0·48 respectively, these corresponding to the percentages named.)

The volume of sand in which the voids will be quite filled with 2 cubic feet of cement paste (made with one bag of cement weighing 204 lb. net) will equal $\frac{2}{\cdot 38 + (\frac{1}{10} \times \cdot 38)}$ $= \frac{2}{\cdot 418} = 4{\cdot}785$ cubic feet $= 8{,}268$ cubic inches. The internal dimensions of a cube having this capacity is arrived at by finding the cube root. Thus $\sqrt[3]{8268} = 20{\cdot}22$ in. cube, say $20\frac{1}{4}$ in. = the internal dimensions of the sides of the gauge box for measuring the sand.

The volume of cement and sand mortar produced is found by $Vc + (Vs \times (1 - P))$: where Vc is the volume of cement paste in cubic feet, Vs the volume of sand in cubic feet, and P the percentage of voids in sand. Therefore the volume of mortar produced will be $2 + (4{\cdot}785 \times (1 - {\cdot}38)) = 4{\cdot}966$ cubic feet.

The volume of aggregate for the voids to be filled with this quantity of cement and sand mortar will be $\frac{4{\cdot}966}{\cdot 48 + (\frac{1}{10} \times \cdot 48)} = \frac{4{\cdot}966}{\cdot 528} = 9{\cdot}405$ cubic feet, or $9{\cdot}405 \times 1728 = 16{,}251$ cubic inches; for this there will be required a gauge box $\sqrt[3]{16251} = 25{\cdot}33$ inches cube, say, $25\frac{5}{16}$ in. in width, depth, and length, internal dimensions, for measuring the aggregate.

The resulting concrete would be of maximum density, after which increase of strength would be directly attained by increasing the volume of cement paste. Assuming that the aggregate had been more perfectly graded and the voids had been 40 per cent., the volume of aggregate required for one bag of cement would then have been $\frac{4{\cdot}966}{\cdot 40 + (\frac{1}{10} \times \cdot 40)} = \frac{4{\cdot}966}{\cdot 44} = 11{\cdot}28$ cubic feet for the same volume of cement paste, thus increasing the volume of concrete 1·885 cubic feet for 2 cubic feet of cement paste, and yet maintaining practically the same strength.

The weights of the bags of cement should be checked to obviate risk of error.

Specifying Proportions.—In days gone by, concrete was specified as 4 to 1, 5 to 1, etc., the sand and coarser aggregate being measured together, and leading to very irregular mixtures and results that could not be relied upon. All experienced architects and engineers now specify the sand and coarser aggregate separately. Thus, a mixture of 1 : 2 : 4 means 1 part of cement to 2 parts sand and 4 parts of coarse material of various sizes.

GAUGE BOXES AND MEASURES

Aggregate Gauges.—Gauge boxes consist merely of four sides fastened together. They have no bottom, so that when they are filled and levelled off at the top, the gauge can be lifted off, leaving the measured materials in a heap. They should not be made too high, and their capacity should not be so great as to make them unwieldy. Some form of handle should be attached. A good arrangement is shown at Fig. 7, where the handles are placed midway at the sides. This arrangement admits of the gauge box being used either end up. Projecting handles at the four corners are objectionable as being in the way of the workmen, and therefore liable to get broken off. The top and bottom edges should have strips of hoop-iron nailed along them to resist wear. It is convenient to make the capacity some definite measure, so that it can be used for measuring a cubic foot or multiples of a cubic foot, or so that it can be used in connection with the scientific system of proportioning already described.

A gauge of $\frac{1}{2}$ cubic yard capacity is a convenient size in general concrete work, and if the height is fixed at 1 ft., allows a fairly large surface for spreading the cement after the gauge has been slightly lifted; the final removal of the gauge is almost the equivalent to one turn over. The following example shows how to arrive at the

dimensions of $\frac{1}{2}$ cubic yard gauge 1 ft. in depth :—

$$\frac{13\frac{1}{2} \text{ cub. ft.}}{1 \text{ ft.}} = 13\tfrac{1}{2} \text{ ft. super., then } \sqrt{13\tfrac{1}{2}} = 3 \text{ ft. } 8\tfrac{1}{10} \text{ in. nearly.}$$

So that a gauge 3 ft. $8\frac{1}{10}$ in. by 3 ft. $8\frac{1}{10}$ in. by 1 ft. deep = $\frac{1}{2}$ cubic yard, and a gauge of

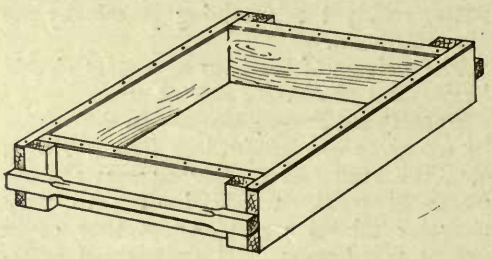


Fig. 7.—Gauge Box with Fixed Handles

the same dimensions top or bottom area, but 2 ft. deep = 1 cubic yard. Care should be taken to make the gauge accurately square, and to provide that it shall maintain this form, or its capacity may become seriously diminished. The joints in the sides should be matched or grooved and tongued together, and the inside should be rough-planed.

Cement Gauges.—A wooden gauge for cement may resemble Fig. 8. It should be made as light as possible consistently with strength, and furnished with some means of transporting it when charged, the cement store being often unavoidably some distance from the mixing platform. As illustrated, the arrangement for conveyance takes the form

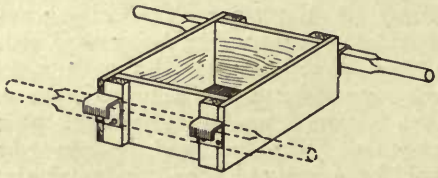


Fig. 8.—Cement Gauge

of two loose carrying bars, fitting under wrought-iron clips or brackets, fixed to the cleats at the sides with stout ironwood screws. This gauge should be accurately square, and have a bottom to it.

Metal gauges for cement are lighter and handier than wooden ones; but cement gauges are not necessary when the whole system of proportioning is based on the weight of cement contained in one bag. The best method of apportioning cement is by weight, and not by cubic measure.

Water Gauges.—If the water is not measured accurately, defective concrete will be produced. Odd buckets and cans should be avoided as tending to errors, which, occasionally, become very serious, and only those measuring appliances should be used that have been carefully examined and certified by some responsible person. For small quantities of water a conveniently-sized can or bucket, as tall and narrow as possible, may be used, providing that it has had its exact capacity or some distinguishing mark legibly painted on it. For larger quantities, a cistern holding just the amount of water required for one batch of concrete should be placed above the mixing board or machine. This cistern should be provided with an outlet pipe and tap as near to the bottom as can be arranged. A short piece of rubber hose attached to the outlet pipe is often a great convenience. All buckets, cans or cisterns should have, legibly painted inside them, a mark to which they are to be filled. This mark should be as near as possible to the top of the vessel, as the risk of adding too much water is thereby reduced. If the quantity of concrete is very large, it is more convenient and accurate to have a cistern fitted with an overflow pipe, and to run water into the cistern until it begins to overflow; the disadvantage of this arrangement is that it is troublesome to adjust the overflow pipe in the first instance. Some firms have found tanks operated by siphons to be convenient and accurate; a cistern controlled by a ball-cock is used to fill the measuring tank, and the latter is emptied siphonically with such suddenness that the amount of water introduced by the opening of the ball-cock is too small to cause any appreciable error. Where variable quantities of concrete are to be mixed, a cistern with its capacity at different levels painted on it should be used. All the water to be used for one batch should be held at one time in the water gauge.

MIXING CONCRETE BY HAND

There are but few hard-and-fast rules with regard to the mixing of concrete. Most foremen and clerks of works favour a particular system of their own, and as long as the result in all cases is a properly mixed material the quickest method is the best. On no account must the concrete be worked after setting has begun. In a method that was common until a few years ago, the materials were measured out (on a 10 ft.

square platform) in a heap on the top of one another, the cement being added last from a wooden box. The heap was then turned over with shovels, one man having an iron prong; the water was applied gently, so as not to wash the cement away. The concrete was turned often enough to obtain a thorough incorporation of the ingredients.

In another method, the aggregate was placed in a layer from 8 in. to 12 in. thick over a platform, the smaller pebbles being at the bottom. The cement was then spread as uniformly as possible over the whole, the materials being then mixed by four men, two with shovels and two with hoes, the former facing each other, and always working from the outside to the centre, then stepping back and going over it again in the same way; the operation was continued until the whole mass was turned. The heap was then turned over again in an opposite direction, the surface of every pebble thus being covered with cement. Two turnings usually sufficed to make the mixture complete.

A method that once had the approval of engineers is to spread out the sand over the platform to the depth of a few inches, and over this to spread the cement. A hole is made in the middle by means of a hoe, and the water poured in, the whole being then mixed by means of shovels and hoes to form a thin paste. The aggregate straight from the washing, or purposely wetted, is added, and the whole mixed. This is a bad method; unless the mixing board has a fillet all round it, there is a risk of losing much of the fine stuff, and the method is slow, giving rise to the temptation to scamping.

A better method is first to spread out the aggregate, then the sand over it, and lastly the cement over the sand. Two shovellings will then mix the material fairly well. It is usual to measure the aggregate in a gauge box, smooth out the aggregate so that the sand gauge may go on top of it, and then fill the gauge with sand. In some cases a slight allowance is made for the sand which will fall into the voids of the aggregate, but this is scarcely necessary if the top of the heap of aggregate is smoothed fairly well. Lastly, the cement is added from a wooden or metal measure, or possibly straight from the bag. Preferably, the water should be supplied through a rose, as the more gentle the application the better. The water is applied during a third shovelling, following which may be a fourth, or the concrete may be shovelled direct into the barrows or other conveyers.

The Association of American Portland Cement Manufacturers recommends the following method: the sand is spread over the board in a layer 3 in. or 4 in. thick, and over it is spread the cement. Two men start mixing the sand and cement, each man turning over the half on his side, starting at his feet and shovelling away from him. In turning the shovel, the materials must fall off the end and sides so that the materials are mixed as they fall. The mixed sand and cement are spread out carefully, and the gravel or stone measuring box is placed beside it, filled, lifted off, and the gravel is shovelled on top of the sand and cement, spreading it evenly. With some experience equally good results can be obtained by placing the gravel measuring box on top of the carefully levelled sand and cement mixture, and filling it, thus placing the gravel on top without an extra shovelling. Add about three-fourths the required amount of water, using a bucket and dashing the water over the gravel on top of the pile as evenly as possible. The men turn over the materials in much the same way as they did the cement and sand, except that, instead of shaking the materials off the end of the shovel, the whole shovel load is dumped and dragged back towards the mixer, so that the wet gravel picks up the finer material. Water is added to the dry spots as the mixing proceeds until the allowed quantity has been used. The mass is turned over again, and if it is streaky or shows dry spots it must be turned again. After the final turning it is shovelled into a compact pile.

"Dry" Mixing.—The so-called "dry" method of mixing is of interest to the concrete block maker who desires to remove the shaped block from the block-making machine with as little delay as possible. In the United States, too, it is used in cases where the contractor is working on a time limit, and wishes to remove the centering, etc., at the earliest possible moment. By this method, no more water is used than will hold the ingredients together when some of the material is taken in the hand and squeezed. Even such concrete as this will show moist on the surface after tamping. The concrete attains its strength more quickly by this method than when it is made really wet, but there appears to be no difference in results as

regards strength when samples of the two kinds are tested after an interval of a couple of years.

MACHINE MIXING AND MIXERS

The popularity of reinforced concrete has been the underlying cause of the attention now being paid to the design and construction of concrete mixers. The necessity of producing a concrete of high quality, and in which the whole of the surface of the aggregate is coated with cement, has had the effect of directing inventive faculty towards the provision of a machine to do the work more efficiently than is possible when shovels are used. A certain knack is necessary in hand mixing, and it is not all labourers that possess it; then, too, the mixing is heavy work, and is likely to be scamped when opportunities for doing so arise. The introduction of the mechanical mixer was the occasion for much controversy, which has long since been settled in its favour; only now and then, as in the case of an important building at Westminster erected in 1910–12, do these responsible prefer hand mixing.

In selecting one of the forty mixers obtainable, the question is not so much nowadays whether it mixes concrete efficiently, but how long the operation takes. To the time employed in the actual mixing must be added that for charging and discharging, but there is no doubt that the employment of any of the well-known machines now obtainable is sure to save time and money. Machine mixing undoubtedly exercises economy in the most costly of the three materials—namely, cement—inasmuch as a given amount can be incorporated efficiently with a larger amount of sand and aggregate than is possible with hand mixing.

Choosing a Mixer.—The concrete mixer has to be designed and constructed to work under very severe conditions, the mention of some of which will suggest to the practical man the points to be borne in mind when choosing a machine. It must be simple, and have as little mechanism as possible, as it will be tended, not by mechanical engineers, but chiefly by labourers, and any delicate mechanism would soon be clogged by the cement. It must be strong, because it has to accommodate heavy batches, and the metal in contact with the concrete must be thick and hard, since the aggregate has great abrasive properties. All parts subject to wear by the concrete should be easily renewable, and the makers should be asked whether or not they can replace from stock the parts in question. It must be compact and eminently portable, because it may have to be used on a number of floors in the same building, and thence may require to be carted miles away to another job. In its design there must be a complete avoidance of arrangements which theoretically are good but practically are bad; in other words, the machine needs to have been designed by someone with practical experience in mixing concrete, as the problem is a different one from that which the designer of mixing machines for soft goods is required to solve. Not only the actual mixing, but all the accessory processes must be executed speedily; it must be easy to charge the machine, and a matter of a few seconds to empty it. It must do its work thoroughly, by which is meant that the cement must be uniformly distributed throughout the sand and aggregate, every particle of which must be coated.

Concrete is not an easy material to mix, each of the three ingredients presenting difficulties of its own. The cement is liable to bridge in a tapered hopper; its dust flies about and clogs any mechanism present; if the machine is not cleaned out regularly the cement will be retained in angles and corners; and a moist cement mixture does not work freely, the trouble increasing as the proportion of cement increases. It might be thought that if cement and sand were run into a revolving drum and this speeded up to a high velocity, the maximum of efficiency would be obtained, but it is not so, as at a certain velocity the cement flies to the side and clings there. Sand, if at all damp, refuses to run unless the shoot or hopper is steeply inclined (experience shows that the inclination must be at least 30 degrees to the vertical). The difficulty in dealing with the aggregate is, first, its abrasive action, which rapidly wears out soft metal; and, secondly, the trouble caused by stones becoming wedged between two rotating arms or between an arm and the side of the mixing vessel; the latter trouble is not apparent in most of the tumbling barrel mixers.

Types of Mixers.—Coming now to details, two broad types of mixer are at present in use, respectively (1) the batch or intermittent mixer, which mixes at one

time any quantity up to the full capacity of the mixing vessel; generally it is a rotating drum, to the inner sides of which are riveted blades which assist the mixing action; a lesser-used type has a pan, either revolving or stationary, with paddles or

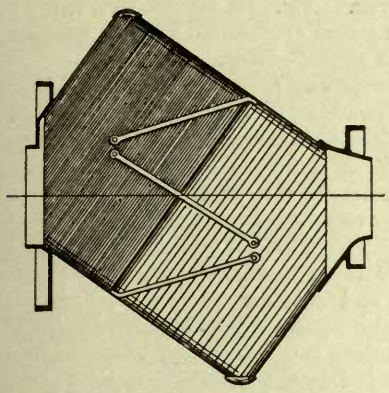


Fig. 9.—Sectional Diagram showing Principal of Chicago Cube Mixer

stirrers to agitate the material. (2) The continuous mixer, which delivers a continuous stream of mixed concrete; the most popular type consists essentially of an inclined cylinder with suitable mixing or agitating appliances.

The method of measuring or gauging the materials is of the greatest importance in connection with machine mixing. In the case of the batch mixer the measures are boxes, hoppers, or even the wheelbarrows in which the material is conveyed to the machine; but the most convenient form of measurer is a skip which may be lowered to the ground to receive the charge, and then in a few seconds elevated and tilted so that its contents pour into the mixing drum.

As regards continuous mixers, owing to the nature of the mixing process the materials have generally to be measured and roughly incorporated in their dry state before they are introduced into the machine; or, as an alternative, there is, as a part of the machine, the necessary mechanism for proportioning the ingredients. Examples of machines so equipped are illustrated later.

There is a growing consensus of opinion in favour of the batch mixer for reinforced concrete work, while the other type is found to have advantages in connection with large works, such as harbours, coast defences, etc., where the concrete requires to be deposited in bulk, and where, possibly, immense quantities are required in one place. The preference given to the batch mixer is due to a number of reasons: (1) It lends itself to the intermittent nature of the work. (2) It is thought that the clerk of works can exercise a more careful supervision over the proportions of the ingredients, but whether as a matter of fact he can do so in practice is a moot point, as obviously in a building of any size he has other things to do than watch a mixer at work for several consecutive hours. (3) Both the proportions of the ingredients and the duration of the mixing can be easily varied, whereas it is a matter of difficulty in some continuous mixers to make any appreciable alteration in the duration of the mixing without interfering with the efficiency of the product, as any slowing down or speeding up of the machine often has the effect of necessarily altering the proportion of water admitted to the mixture. The continuous mixer has a different mixing efficiency with different classes of aggregate, and, theoretically, with any increase in the proportion of sand the inclination of the cylinder should be increased, this not often being possible. (4) The batch mixer enables the ingredients to be mixed dry, if required.

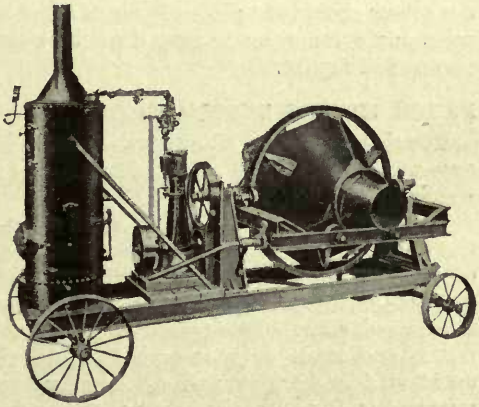


Fig. 10.—Chicago Cube Mixer with Engine and Boiler

The power for driving a mixer may be either hand or engine, and in the majority of cases the larger mixers are self-contained, a boiler and engine, an oil engine or petrol motor, or an electric motor to be connected up to the supply mains, being mounted on the same base as the mixer. Naturally, the

power-driven machine has a much greater capacity than the hand-driven one. A warning with regard to advertised capacities may here be given. Makers often state most definitely the weight or the number of cubic feet of concrete which their machines can mix in the course of an hour, and they

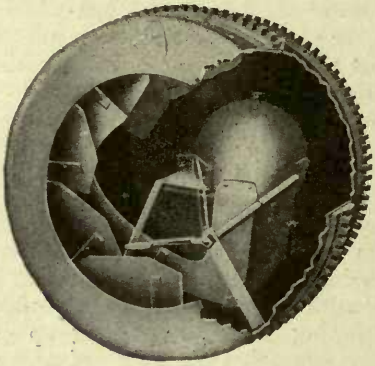


Fig. 11.—"Cut-away" View of Eclipse Mixing Drum

advertise the results of perfectly honest tests; but inasmuch as the capacity of a machine must vary according to individual conditions—the disposition on the site of the cement, aggregate, etc., the nature of the job, and the amount of experience of the operatives—a contractor who is choosing a machine should not place too much reliance upon any comparison he may draw between two sets of figures.

BATCH MIXERS DESCRIBED

Revolving Drum Machines.—There is any number up to a score of these from which to choose.

Chicago Cube Mixer.—This is based essentially on the old-style tumbling drum—a cubical box journalled at diagonally opposite corners—but the shaft in the old-style mixer is replaced with hollow trunnions riding on rollers and made large enough to serve as openings for charging and discharging the mixer (*see* Fig. 9). To rotate the cube there is a circumferential rack fastened around it on a drum at right angles to, and midway between, the trunnions; this rack, geared with a pinion shaft, is operated by the engine shaft in such a way that there is no gearing in the way of the materials either in charging or discharging. The sharp corners and edges of the cube are rounded off to obviate the possibility of the fine material effecting a lodgment. The manufacturers of this mixer have found that the batch is folded over on itself, and pressed into a contracting space shaped alternately like a wedge and like a pyramid; and on this fact is based their statement that the mixing is done by kneading and not by stirring. Fig. 10 shows the cube mixer, with steam engine and boiler; another type has a petrol motor.

Barker and Hunter.—The special feature of this machine is that it cannot be used as a continuous mixer. It is fed while revolving in one direction, and must be reversed before the mixture can be discharged. The actual mixer consists of two truncated cones, whose bases merge into a cylinder, the ends of the cones being mounted with roller path bearings and rotating on rollers mounted on a suitable frame. Two hoppers are fitted, and the mixer can be charged from either end or from both ends, which are always open, with the cone either stationary or in motion. The makers say that it takes twenty seconds to charge the material into the cone, sixty seconds to mix them completely, and ten seconds to discharge.

Eclipse.—This machine is built shallow so as not to necessitate mechanical charging devices. The drum has a shell of rolled steel plate, and has flanged steel heads; it is supported by trunnion rollers, and is rotated by gears meshing into two gear rings. At the charging end is a large circular opening in the flanged head, and

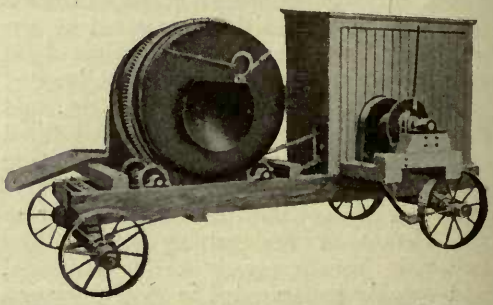


Fig. 12.—Eclipse Mixer with Petrol Motor

diagonal overlapping charging blades extend into the drum. In the interior of the drum are diagonal blades firmly attached to the sides. One blade extends from the charging end diagonally to the discharging pocket shown at the rear of the drum in Fig. 11. The aggregates are carried into the mixing chamber by conveying blades rigidly

attached to the sides and ends, so designed that the material cannot slop out when mixing. The discharging shoot extends into the drum through the head at the discharging end. A shaft extends through the drum near the shoot, and on it is mounted a door held during either mixing or discharging by a strong spring under tension. When the operator throws the door past the centre of tension in either direction, the spring completes the movement and holds the door until it is again held by the operator. Fig. 12 is a special design mounted on a truck and equipped with a petrol motor housed in the casing.

Gauhe.—In this mixer the drum (supported by friction rollers) revolves, while paddles

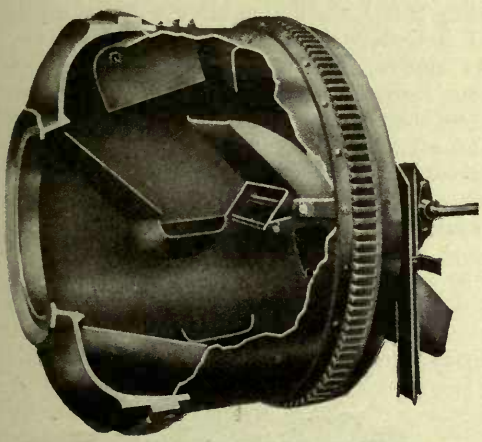


Fig. 13.—"Cut-away" View of Koehring Mixing Drum

and a scraper are fixed and serve to mix the material. The material enters the drum through an opening at one end, and the discharge is through a sliding door controlled by a lever. The elevating device is on the lines of the Fawcett, described later, and therefore, like that machine, it requires considerable head-room. The road wheels are of large diameter, so as to allow trolleys to pass beneath the mixing drum to receive the concrete. The equipment includes an automatic water-measuring tank.

Koehring.—In this the drum is of boiler-plate steel with cast heads, surrounding each of which is a gear ring. The double drive has the advantage that, should a tooth break, the machine can continue on a single drive. An interior view of the drum is given in Fig. 13, whilst Fig. 14 is a photograph of the mixer with steam boiler and engine, side loader, and water measuring tank (shown above the mixing drum); the

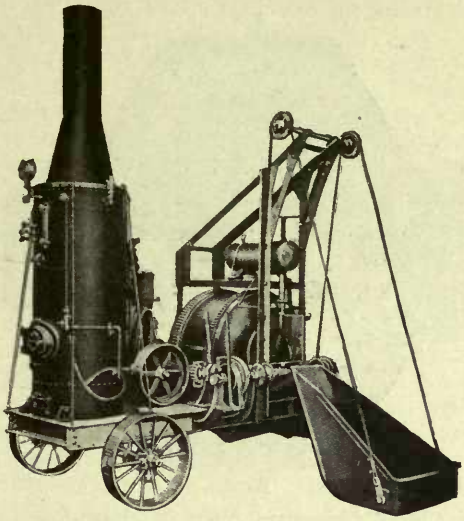


Fig. 14.—Koehring Mixer with Engine and Boiler

operating levers and hoisting mechanism are clearly shown. The discharge shoot is made in two pieces, the outer end being stationary, and the inner end pivotal, so affording ample clearance for wheelbarrows in the discharge, and permitting the inner end of the shoot to assist in the mixing when tilted inwards. The side loader is a short, wide, and low bucket with round corners and sufficiently low pivotal point to require but little power

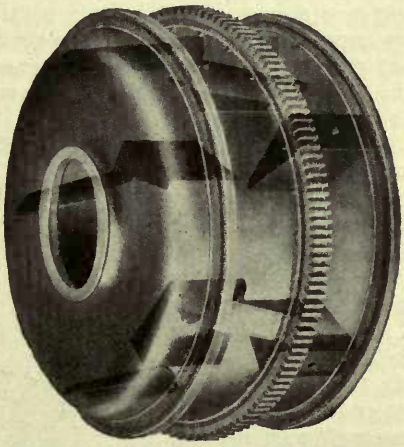


Fig. 15.—Marsh-Capron Non-tilting Mixing Drum

to raise it; it can be elevated to almost a vertical position, so that the material readily flows into the drum.

Marsh-Capron.—Both tilting and non-tilting machines of this manufacture are

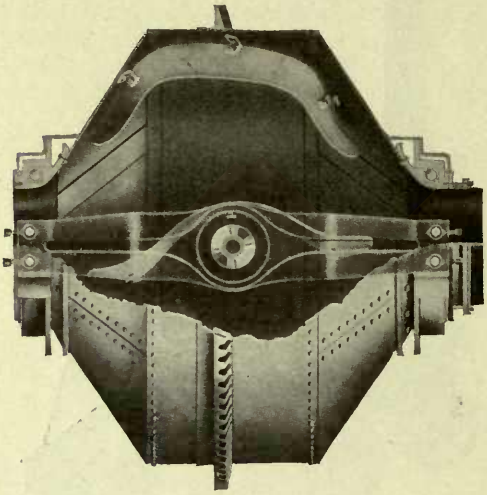


Fig. 16.—"Cut-away" View of Marsh-Capron Tilting Mixing Drum

known, and in each the drum is the special feature. In the non-tilting style, the drum (*see* Fig. 15) is cast in two parts, the connecting flanges fitting one within the other and centering the two halves, which are securely bolted together. The steel mixing blades are bolted to the drum, and both these and the steel mixing buckets are so placed as to give a thorough mixing action to the material, providing an end-to-end as well as a lifting and pouring action. The driving gear surrounding the drum consists of five segments absolutely interchangeable, so that should a tooth be broken a segment can be renewed at the labour of removing and replacing five bolts, the drum being kept in place and the rest of the machine remaining undisturbed. In the tilting machine the drum (*see* Fig. 16) has its blades arranged in serpentine form and attached with brackets, there being sufficient space between blades and drum to allow of thorough washing out. Fig. 17 shows the machine with the drum in the tilting position.

McKelvey.—In this machine the drum consists of a short cylinder with a conical hopper on the feed side and a trumpet-shaped discharge funnel on the other side. It revolves on friction rollers, and is operated by a chain and sprocket ring. Its chief feature is the means adopted for ensuring the proper mixing of the material. The special device used (*see* Fig. 18) is known as a "gravity shovel," and is pivoted to the inside of the drum; the top edge lies against the rising side, collects the materials in that part of the drum, and, after it is filled, the overflow slides rapidly off the lower edge and the balance is carried upward to a point where gravity causes the shovel to cast the material outward and downward, turning it over in the fall.

Messent.—This is based on the old tumbling barrel, the design being a closed vessel of irregular shape revolving on an axle. As used on the Dover Harbour works, it was mounted on a steel-framed carriage, there being one electric motor to rotate the mixing vessel and another to give a travelling motion to the carriage, the materials being thus mixed in course of transit between place of charging and the work.

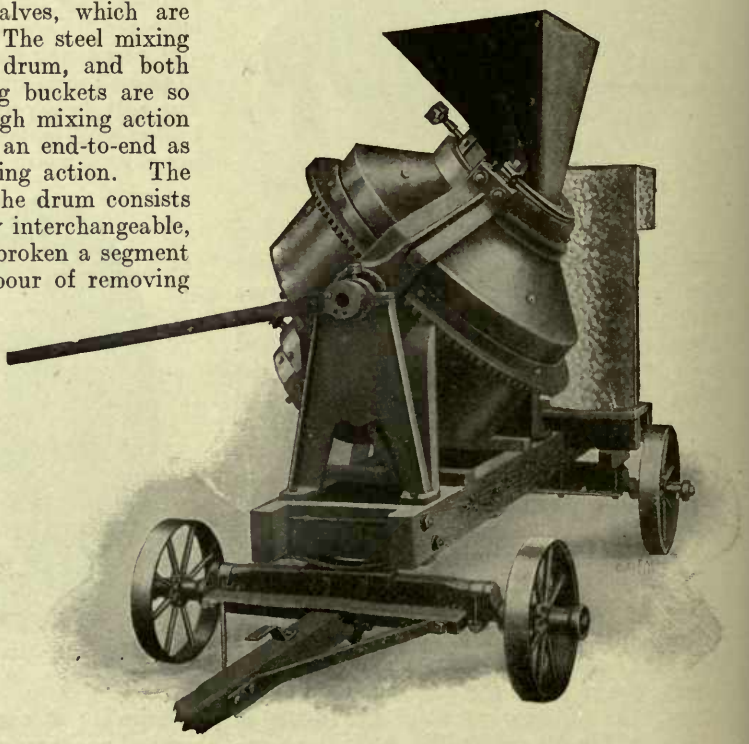


Fig. 17.—Marsh-Capron Mixer, with Drum in tilting position

Pioneer.—This is a small hand mixer, capable of handling between 250 lb. and 350 lb. per batch. It can easily be worked by one man, and is too simple to require detailed explanation, being simply a mixing drum supported on a horizontal spindle. The materials are introduced, the lid closed,

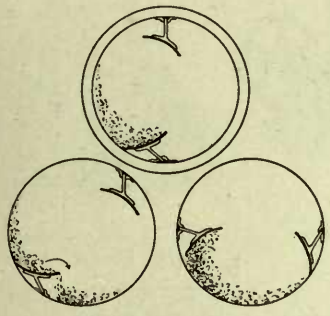


Fig. 18.—McKelvey "Gravity Shovel"

the drum rotated three or four times, then the water contained in the tank at the side is allowed to enter the drum, through the perforated shaft, and the drum is finally rotated again a few times, the mixture then being complete.

Ransome.—Machines of this make largely owe their conception to E. L. Ransome, whose name is of importance in the history of reinforced concrete in the United States. A variety of Ransome machines is made, all incorporating the vital feature—namely, a drum with mixing blades of a special type. Figs. 19 to 24 are photographs of six arrangements of the Ransome mixer, there being an explanatory inscription to each figure. The drum is not of the tilting type, the discharge being made possible merely by opening a shoot. Each type of machine has a hopper which measures the exact amount of aggregate for each batch, there being in addition a water tank which automatically gauges the exact amount of water required. Within the drum is a series of steel scoops (*see* Fig. 25) which plough through the material repeatedly, pick it up, and carry it upwards till it slides out. The aggregate having been delivered from the hopper into the drum, the hopper gate is closed and the hopper re-charged in readiness for the next batch. The elevating hopper is so arranged that it can be lowered until its bottom rests on the ground. When required, a lever is pushed, and the elevator tips the material into the hopper. Fig. 27 shows two elevations and a plan of a Ransome belt-driven mixer.

In Great Britain the Ransome mixers are built by Ransomes and Rapier, Ltd., of Ipswich, and in the United States by the Ransome Concrete Machinery Co., Dunellin. The English and American machines conform to the same general patterns, but there are minor differences of detail; for example, in the American mixing machine the interior of the drum resembles Fig. 26.

Roll.—This has distinct points of novelty (*see* Fig. 28). The drum consists of two cup-shaped halves mounted on a spindle in such a way that they can be drawn apart to discharge the concrete. The drum consists of two castings, to one of which is bolted a sleeve carrying a worm which serves to move one half of the drum along its spindle when discharging is necessary. The mixer is equipped with an elevator consisting of a measuring skip hauled up by a wire rope and returning by gravity.

Smith Hand Mixer.—As shown by Fig. 29, this machine is driven by means of crank handles—one man to each side—but it is easily adapted for power driving. The ends of the drum are sloped inwards until they almost meet, forming two wedge-like chambers united by a $4\frac{1}{2}$-in. slot, extending diametrically across the drum. The drum itself is suspended on chains, four guide rollers being provided to keep it in alignment and prevent it from swinging during the mixing process. The principle of the mixing can be understood by bearing in mind the old-fashioned hour-glass, the material from one chamber having to pass through a restricted opening into the other one. Only two or three turns of the drum are required. The charging platform supplied is only $15\frac{1}{4}$ in. high, thus enabling a wheelbarrow to charge directly into the machine, and the mixer is so constructed that, if it were desired, it could discharge directly into a trench.

Smith Power Mixer.—The drum of this machine consists of two cones united at their bases, as shown in Fig. 30. The blades which assist the mixing are arranged spirally; and at the discharge end are drip rings which prevent the wet material from running down the cone and over the roller tracks. The drums are partially lined; for instance, the large ends of the cones are made of double thickness, because that is where the principal wear comes. This

lining is a regular repair part and is easily fitted. With certain of the sizes a power tilting apparatus is supplied, the makers believing that the tilted drum provides the fastest method of discharging. It is only the smaller sizes of the Smith power mixer that are recommended for reinforced concrete work.

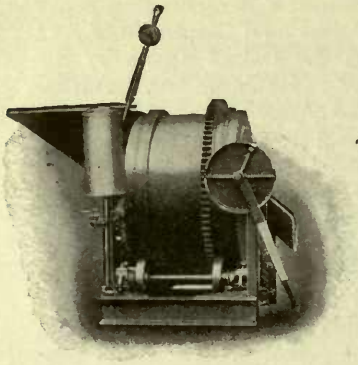


Fig. 19.—Belt-driven Ransome Mixer

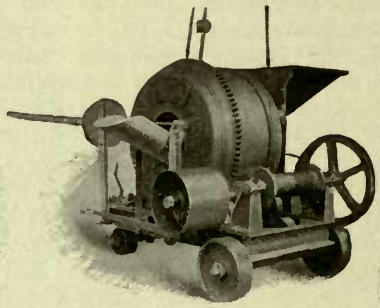


Fig. 20.—Ransome Mixer with Hoist

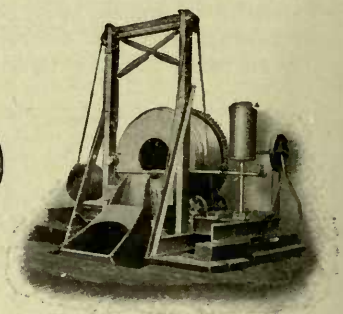


Fig. 21.—Ransome Mixer with Elevating Hopper

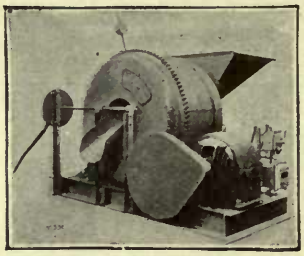


Fig. 22.—Ransome Mixer with Electric Motor

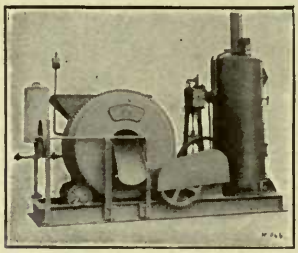


Fig. 23.—Ransome Mixer with Boiler and Engine

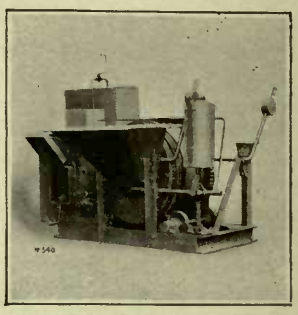


Fig. 24.—Ransome Mixer with Oil Engine

Taylor.—This resembles the Smith in

Victoria.—The drum of this mixer is a cylinder with four deflecting surfaces or inclined planes (*see* Fig. 33). Lifting blades are riveted to the drum and extend from the discharge end well towards the feed end. As the drum rotates, the lifting blades elevate the material, which drops

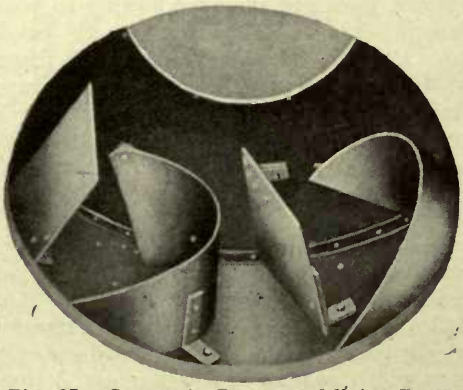
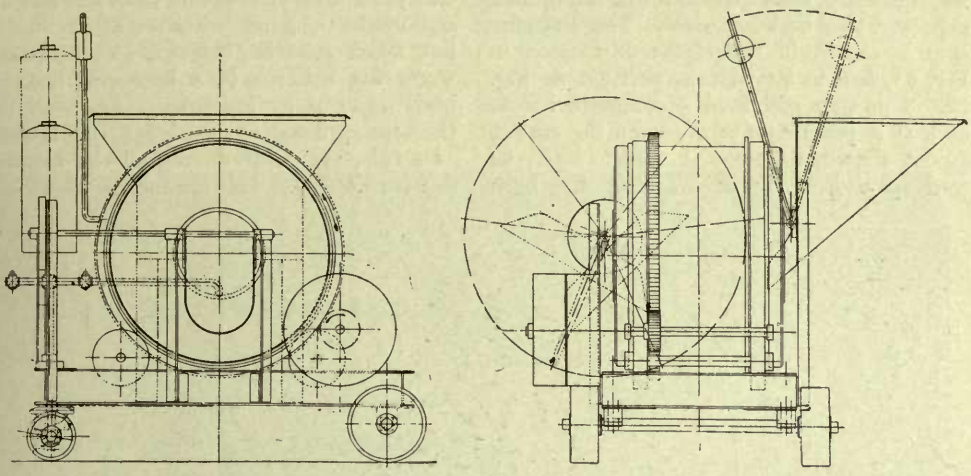


Fig. 25.—Scoops in Ransome Mixing Drum (English)

Fig. 26.—Scoops in Ransome Mixing Drum (American)

upon the inclined planes and is thus thrown across the drum and returned again by the opposing deflectors. The process is repeated twice for each revolution, and submits the mass to twelve distinct mixing actions per turn of the drum. The complete machine, on a truck, with engine and boiler, side loader, and automatic water tank, is shown in Fig. 31. There is a low feed level, which is of especial advantage in a portable plant charged by wheelbarrows. For the purpose of emptying the mixture, a swinging spout, pivoted below the discharge opening, is projected into the drum. It will be understood, on referring to Fig. 31, that the spout is pivoted to the vertical frame shown, and easily swings forwards or backwards. When the spout is inserted into the drum the mass falls from the blades upon it, and the concrete flows into the receptacle prepared for



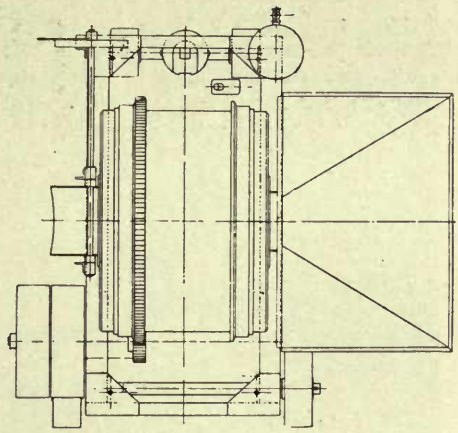


Fig. 27.—Elevations and Plan of Ransome Belt-driven Mixer

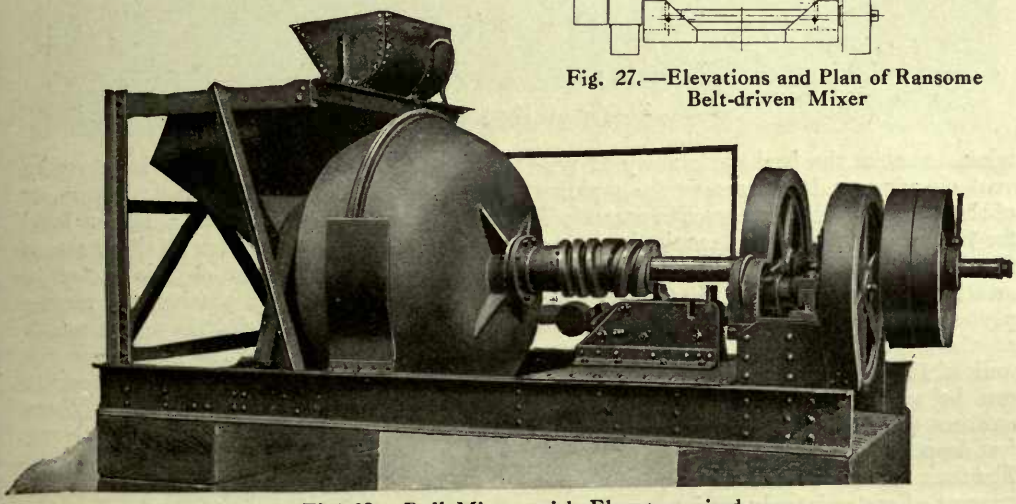


Fig. 28.—Roll Mixer, with Elevator raised

it. The flow is under easy control, as the shoot may be removed from the drum at any moment by a mere turn of the wrist. Any quantity, from a quart to the full mixing capacity, can be withdrawn. The charging skip is shown in the elevated position in Fig. 31, and in its bottom position in Fig. 32. This skip will hold a complete batch, and, as it progresses upwards on the curved guides shown, its nose is thrust into the feed opening of the drum. Care has been taken to make the feed angle steep to prevent clogging, and to increase the rapidity of the feed. The makers provide extensions for the loader frame when required, so that the skip can be lowered to receive the batch on a level below the mixer. In the two complete views a rectangular tank will be noticed at the top. This is an automatic water tank of 18-gal. capacity, with a gauge which can be set so that the tank will supply uniform quantities of water varying from a few drops up to the full capacity of 18 gal. The tank serves as an indicator to show the amount of water introduced into any batch of concrete; for this purpose the gauge is set for the maximum capacity, the tank is allowed to fill, and the valve is then opened. The pointer on the outside will then fall, and a glance at the gauge will show at any instant how much water has run out. The necessary water for a batch in a large-size mixer is best obtained by measuring the water into the mixing drum in two lots; this is safer, as a rule, than introducing all the water at one time. When this automatic water tank is a part of the equipment, it is necessary to supply the water to it under pressure, or from a reservoir placed at a higher level than the tank. The makers fit either steam engine, petrol engine, or electric motor to their mixers, and they particularly recommend the slow-speed electric motor which may be geared direct to the mixer.

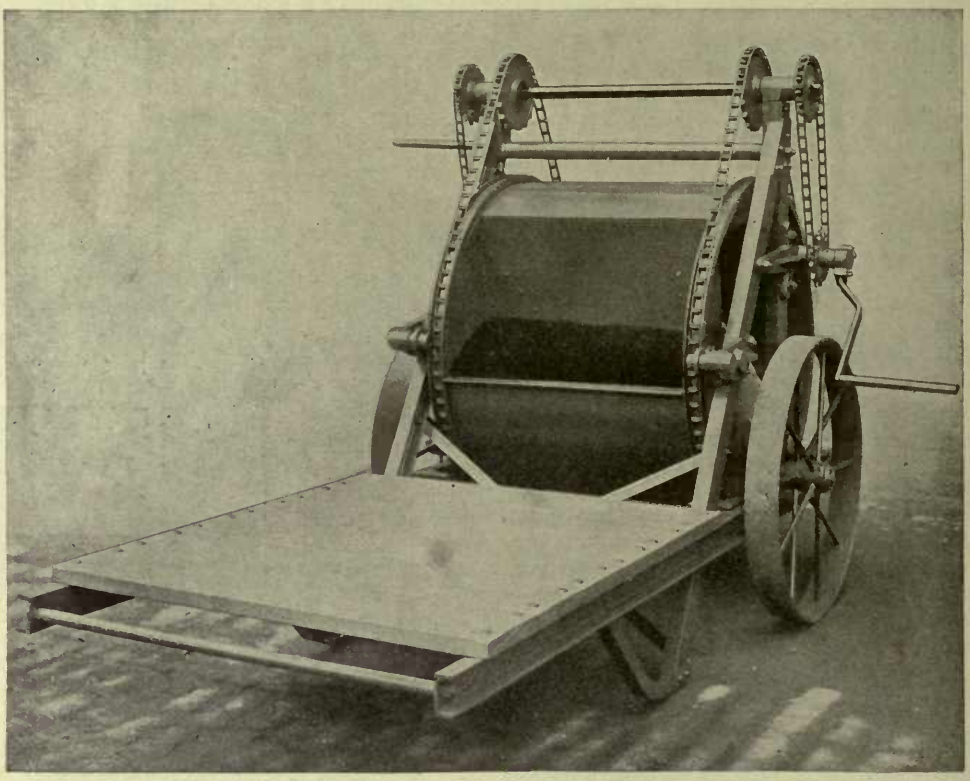


Fig. 29.—Smith Hand-driven Mixer

Other Batch Mixers.—The drum mixer is not the only type of batch machine. There are a few examples of mixers having pans and stirrers.

Express.—This has a stationary pan to hold the material (*see* Fig. 34), revolving

in it being a series of paddles and rakes attached to arms which project from a central power-driven capstan head. By means of two discharge doors in the bottom of the pan the smaller size mixer can be emptied in fifteen seconds, both feeding and discharging taking place while running.

Fawcett.—The material is contained in a fixed drum in which a number of arms or paddles rotate in different directions, the arrangement being such that when the discharge door at the bottom of the drum is opened, the revolving paddles scrape the concrete out (*see* Fig. 35). The mixing is of an efficient character, and the open pan enables the attendant to see when the concrete has attained an even colour throughout. The machine has an elevator for measuring and feeding the charge, and an automatic water tank for measuring the water. The elevator is a skip which, when filled, ascends along a pair of inclined guides by means of a wire rope which is wound on a drum actuated through a friction wheel. When the skip arrives at the top, it tips automatically into the mixing vessel and is then lowered for another charge.

Gaspary.—The hand-driven mixer (Fig. 36) has a trough capable of treating up to 45 cub. ft. an hour, according to size. Above the tilting trough is a small water tank, the water reaching the materials through a perforated pipe. In the power-driven mixing machine, there is a drum with blades which rotate in either direction, this change in the direction being of advantage for cleaning and emptying the mixing drum, and allowing of any stones stuck fast between the drum and the blades being instantly removed. Continuous machines are also made by the Gaspary firm, as noted on a later page.

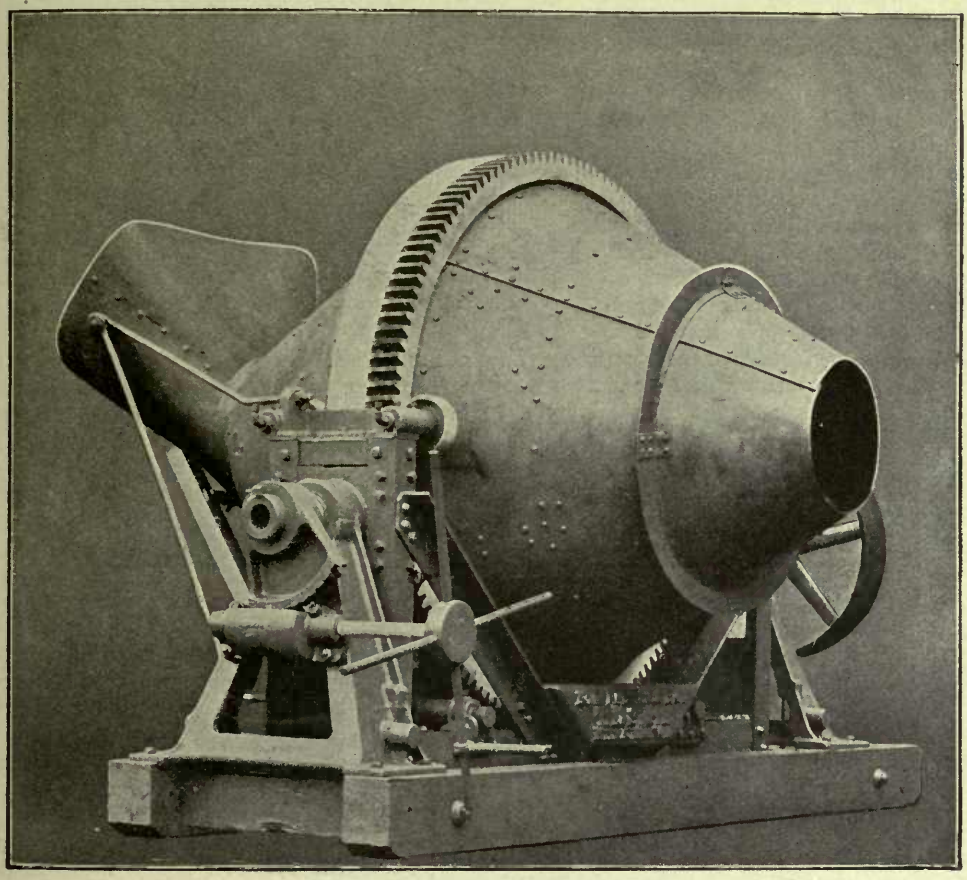


Fig. 30.—Smith Power-driven Mixer

Oehler.—This is a Swiss machine resembling the Fawcett, but the paddles rotate in one direction only.

Open - Drum.—This mixer has a cylindrical drum, normally upright, with conical top which facilitates discharging. The sides of the drum are of steel plate, but the bottom is a casting having a circumferential rack, by means of which the drum is rotated through gearing. The drum is carried by a supporting frame, in which is a ball - bearing for a short vertical shaft which projects into the mixing drum. The mixing is efficient, and is much facilitated by parallel blades riveted to the sides of the drum. The drum, being so near the ground, can easily be charged from wheelbarrows on a loading platform, while for discharging there is a hand or power arrangement for tilting the drum. The capacity varies from $2\frac{1}{2}$ to 18 cub. ft., according to the size and style.

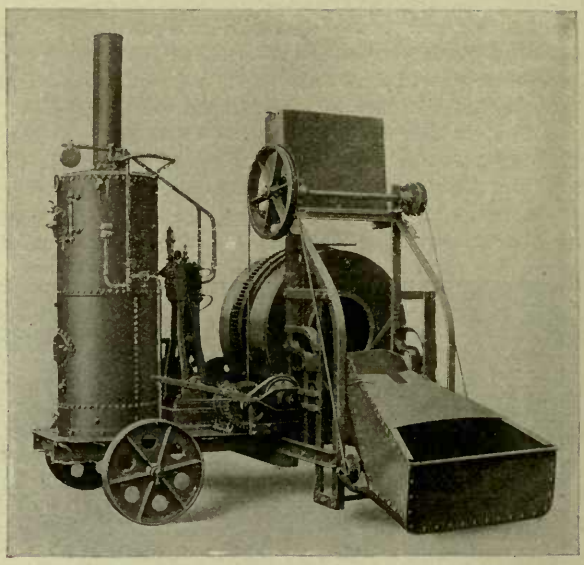


Fig. 32.—Victoria Mixer, with Skip Lowered

Pansy.—As shown in Fig. 37, there is an annular vessel which runs on a ball-bearing cast-steel base, and is driven through toothed gearing. Through the centre of the machine is a perpendicular shaft, to which are hung angle-iron frames, and to these again adjustable ploughs are fixed. The pan revolves at the rate of ten or twelve revolutions per minute, and as the ploughs remain stationary, the materials are thoroughly mixed in a short time. The materials are gauged in the ordinary way on a platform above the machine, and tipped directly into it, being there levelled by a scraper, the ploughs lowered and the material mixed dry. Next, water is added, and the materials further mixed. When mixing is complete, the ploughs are raised, the scraper lowered, and, by means of the trapdoors in the bottom, the mixed concrete passes into a receptacle prepared for it. The doors close automatically by means of a special catch, and as soon as the material is all discharged the pan is ready for another batch.

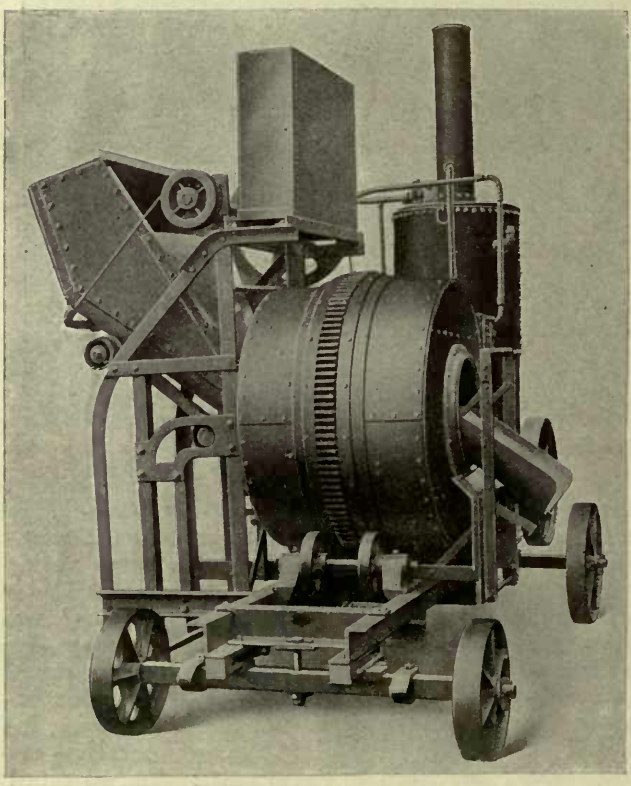


Fig. 31.—Victoria Mixer, with Skip Elevated

Whalley.—In this machine the materials are discharged from a

hopper (*see* Fig. 38) into a stationary pan and the mixing is done by stirrers mounted on arms projecting from a vertical shaft. Each stirrer comprises a vertical cutting part and a scraping part, the latter being almost in contact with the bottom of the pan. The stirrers tend to force the material towards the sides, and when the mixing has been completed a door is opened and the mixture discharged simply by the continued rotation. The parts of the machine liable to wear or replacement are the bottom portions of the stirrers, and they can be readily renewed.

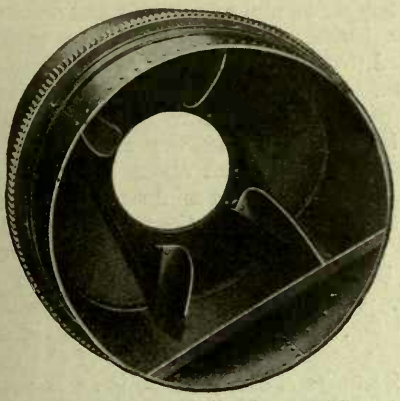


Fig. 33.—"Cut-away" View of Victoria Mixing Drum

CONTINUOUS MIXERS

Non-proportioning Machines.—These are of much simpler construction than the mixers fitted with proportioning or measuring arrangements, and a few examples will now be briefly described.

Gaspary. — Only the smallest of the Gaspary continuous mixers need be mentioned here. Fig. 39 shows a power-driven machine, which, however, can be adapted for hand use, and which may be either movable or stationary according to requirements. The materials have to be fed in measured quantities, the water being obtained from a supply tank shown above the mixing cylinder, there being a perforated pipe extending from the tank right into the cylinder. The dry mixture enters the cylinder, is wetted and mixed, the mixing action being assisted by shovels which are turned by the rotation of the cylinder.

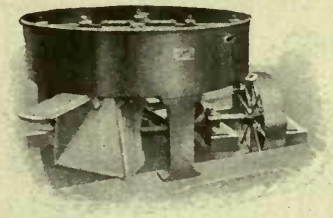


Fig. 34.—Express Mixer

Mason.—The feature of this machine is a cylinder slightly larger in diameter at one

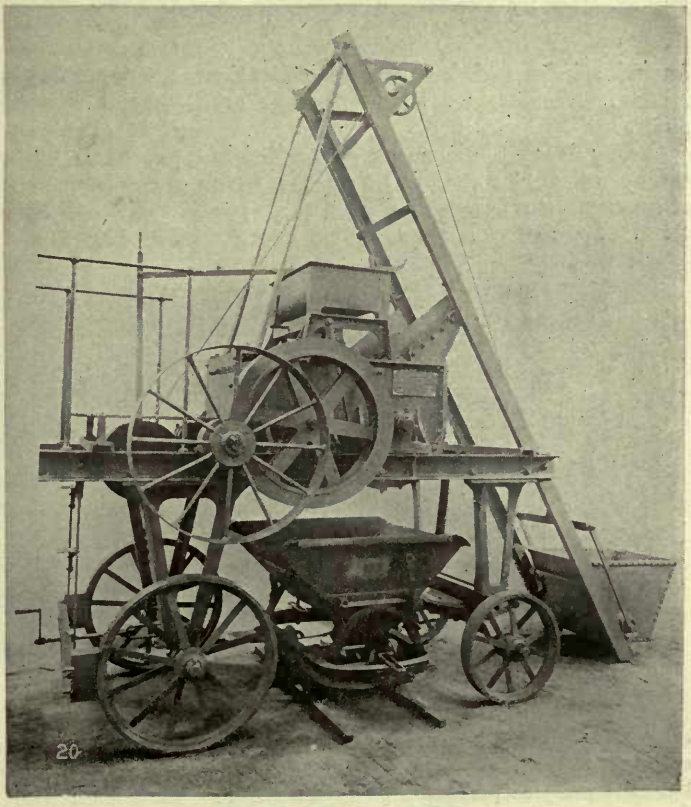


Fig. 35.—Fawcett Mixer

end (the discharge end) than at the other where the feed hopper is situated (*see* Fig. 40). The cylinder is mounted on a

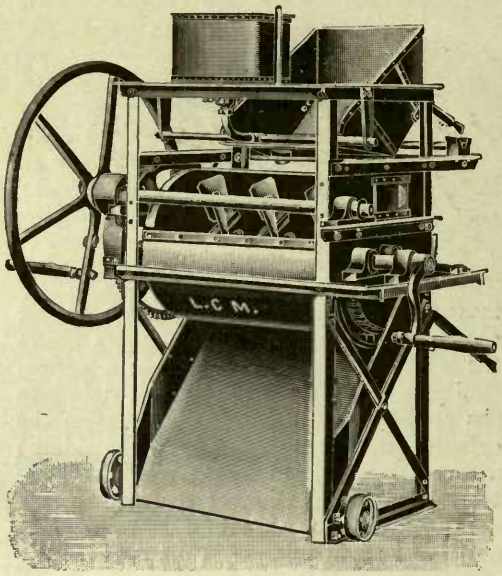


Fig. 36.—Gaspary Hand-driven Tilting Trough Mixer

central spindle and has ribs to assist the mixing. Machines for hand or power are made.

Gravity Mixers.—No power is required in the operation of a gravity mixer, the force of gravity alone being relied upon to do the work. The materials are shovelled into the top end of a vertical or inclined shoot, a common pattern of which has a large number of round pins penetrating the shoot from front to back, there being in addition a number of baffle plates. The rows of pins are staggered with respect to the rows immediately above and below them, and it follows that when the materials fall from the hopper on and off the pins and deflectors, they become incorporated. The water is added (by means of a flexible hose) half-way down, this giving the materials a chance to get partly mixed before being wetted.

The Owens is a gravity mixer in which, there is a steel shoot about 7 ft. long containing three sinuous mild steel bars extending in one length from the top to the bottom. The bars are fixed rigidly at the top, but at the bottom are held loosely by means of a ½-in. pin, which passes through them. In addition, there are eight large and six small baffle plates fixed to the sides of the shoot. To facilitate cleaning, there is an inspection door. The water supply pipe is fixed behind the lower edge of the top baffle plate, and is perforated both back and front, the whole of the baffles and bars receiving a spray of water. The materials, having been introduced in any convenient manner, fall through the shoot and are alternately split up into two columns and then united, this action being repeated eight times.

Proportioning or Measuring Machines.—Brief particulars will now be given of eight examples of continuous mixers with provision for proportioning the concrete ingredients.

Bolte.—This is an American machine (*see* Fig. 41) adapted for hand or power, according to size. The principle of this machine is the conveying of proportioned quantities of cement and aggregate, by means of a rubber belt, into a mixing cylinder, where

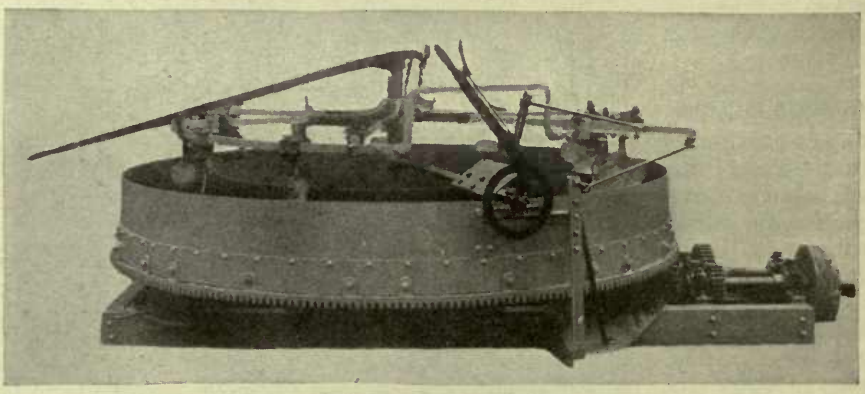


Fig. 37.—Pansy Mixer

the material is agitated by six blades. The hoppers are shown in Fig. 41*a*. The belt forms the bottom to them and draws off the materials through adjustable gates, discharging them into the cylinder, where, 15 in. from the lower end, the water is introduced by means of a perforated pipe connected to the water tank above. As it is well known that cement and sand do not readily flow at all times, the hoppers are fitted with agitators, the cement agitator being worked by a cam on the rear belt shaft and the sand agitator consisting of a worm on a continuation of the cylinder shaft, this being clearly shown in Fig. 41*a*. These mixers are essentially portable, and can be taken from floor to floor as the work proceeds.

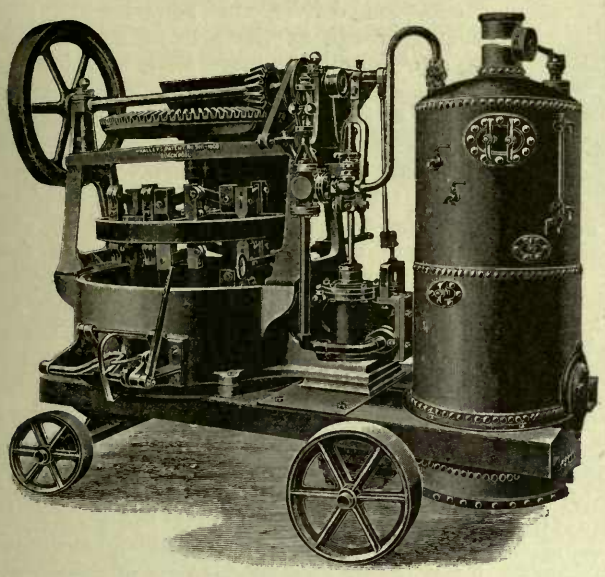


Fig. 38.—Whalley Mixer, with Engine and Boiler

Coltrin.—Machines of this make are made in eight sizes, between which there are various differences of detail, but for the present purpose the No. 14 machine (Fig. 42) may be taken as representative. It has three hoppers or pockets for automatically proportioning aggregates, sand, and cement, and the proportioning device works without the use of gears, sprockets, or chains. The pockets on each side are operated by one sliding belt, which moves forward and backward on rolls and acts as a plunger for delivering the material. The sliding part is moved by two crank arms attached to the square shaft, which also passes through the cement hopper, where its reciprocating action prevents any bridging of the cement. Each material can be proportioned to a nicety, and the materials can be fed in from one or both sides. The mixing device is a special feature, and unlike any other mechanism described in these pages. It consists of seven steel blades twisted spirally and bolted at each end to a solid casting, as shown in Fig. 43. This construction cuts through, lifts, and pours the material, the result being a most efficient mixing. Between the engine and the mixer is a friction clutch, which, besides being generally useful, has one special advantage: should a large stone get into the mixer accidentally and bind

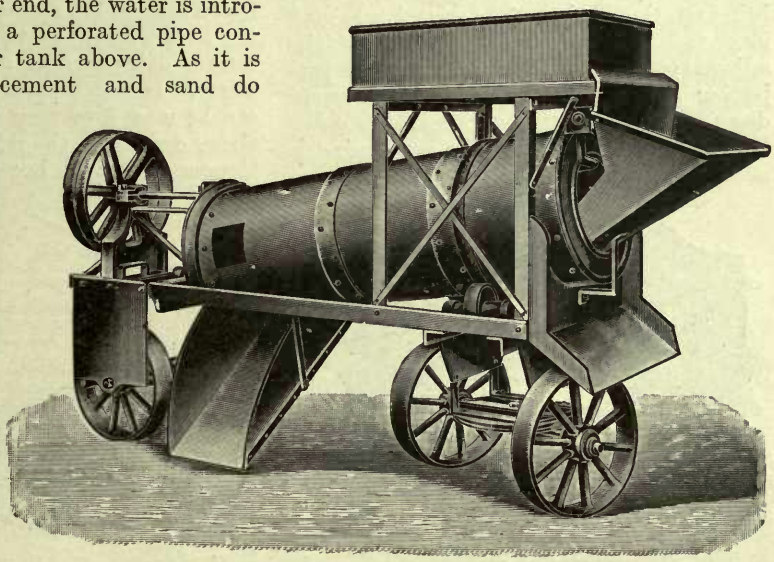


Fig. 39.—Gaspary Drum-type Continuous Mixer

between the blades and the vessel, the clutch will slip, so avoiding breakage of the mixer. The water tank with special pro-

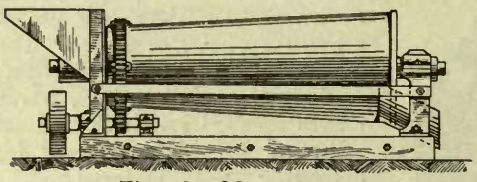


Fig. 40.—Mason Mixer

portioning arrangement is furnished with all machines for use where regular direct pressure is not available.

Carey-Latham.—In this machine, chain and bucket elevators are employed, one for the sand and another for the aggregate, to feed the mixing cylinder, which is inclined towards the discharge end at an angle of 8 degrees, revolves on friction rollers, and has projections on its inner surface to assist the mixing. The cement is fed to the cylinder by means of a worm. Other types of machine bearing this name have been used, but the above is the most generally useful.

Gaspary.—The funnel-dish mixer made by this firm has a number of funnel-shaped reservoirs open at the bottom, below them being rotating blades. The materials are fed into the reservoirs, passed through the adjustable openings, dropped upon the projecting rims of the blades, and are swept down into a mixing trough, in which a screw is working and in which the moistening is effected.

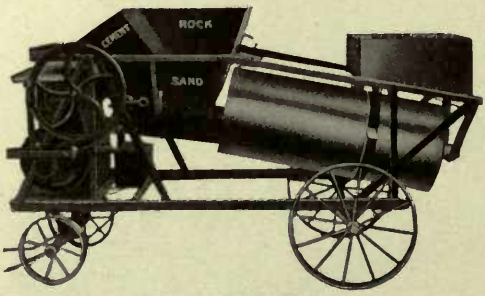


Fig. 41.—Bolte Mixer

Kent.—A variety of machines are made by this firm, the principal being a three-hopper machine in which very carefully designed arrangements are employed to proportion the materials accurately. Special devices have for their purpose the prevention of bridging of the sand and other materials, without employing agitators. The appearance of a typical machine is shown in Fig. 44. The three hoppers with their feed mechanism deliver the materials to the mixing trough at the rate of from 35 to 40 discharges per minute for each material. The sand and cement drop in the same place, and the dry mixing begins immediately and continues for more than half the entire length of the trough. Two water sprays are provided, one spraying directly into the trough, and the other into the aggregate before it is discharged into the dry mixture of sand and cement, the object being to cause the fine stuff to adhere to the aggregate instantly. Each spray has a separate regulating valve. Passing through the cylinder is a square shaft to which numerous paddles are bolted.

Nims.—In this machine, the mixing vessel is of cast iron in one piece, shaped something like a pair of cubes interpenetrating diagonally and revolving on the long axis. A belt delivers the materials, which have been proportioned in a special form of measurer, to the mixer. The mechanism for measuring consists of a wheel 15 in. in diameter and 11 ft. long, revolving under a hopper which is 12 ft. long. In the wheel are receptacles which receive the materials as they fall from the hopper, and, as the wheel continues to rotate, deliver them to the belt conveyer below. In the hopper are partitions which can be moved to vary the proportions. As above described, the mixer acts on the continuous principle, but by fitting an auto-

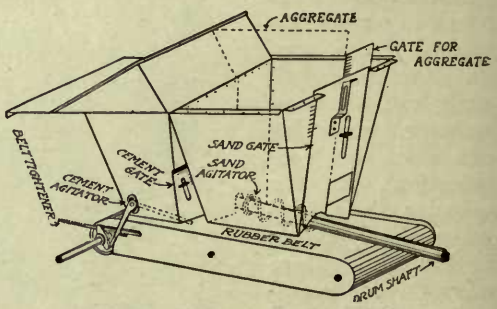


Fig. 41a.—Hoppers of Bolte Mixer

matic gate to the discharge end, it can be used as a batch mixer.

Perfect.—As shown by Fig. 45, there are in this mixer three distinct hoppers, in connection with each of these being a revolving feed drum with deep flanges at each end,

and a regulating gate for proportioning the material set by a lever moving over a graduated quadrant. The feed drums make only one or two revolutions a minute, and a regulator strikes off the cement and allows a fixed quantity to pass as a layer of uniform width and thickness. The stream of cement combines with a stream of sand, and then falls on a stream of aggregate, passing thence to the mixing cylinder, in which is an adjustable water spray.

Trump.—This type of machine requires but little power, as gravity to some slight extent is utilised. The measuring device consists of three cylinders for the sand, aggregate, and cement respectively, and from this the materials pass to three rotating tables, where they meet fixed knife edges that scrape the materials off into a trough below in required proportions. The set of the knife can be altered to vary the proportion. The principle of the measuring device is clearly illustrated by Fig. 46.

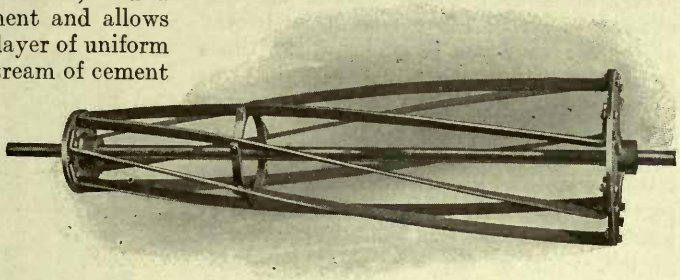


Fig. 43.—Coltrin Mixing Blades

The materials, having been proportioned, flow together in the form of one stream (*see* Fig. 46), which falls into a cylinder, where the materials are moistened by water sprayed from a perforated pipe. In the cylinder, steel paddles rotate on a shaft and complete the mixing.

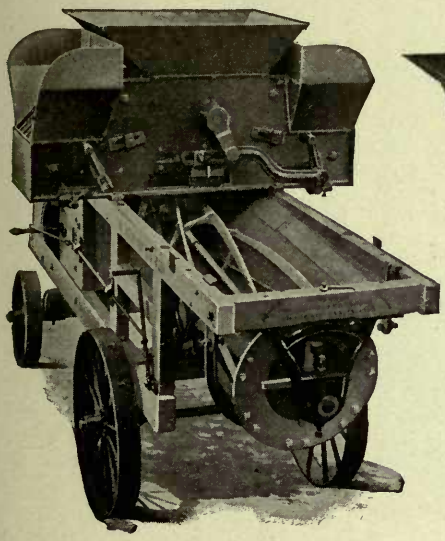


Fig. 42.—Coltrin Mixer

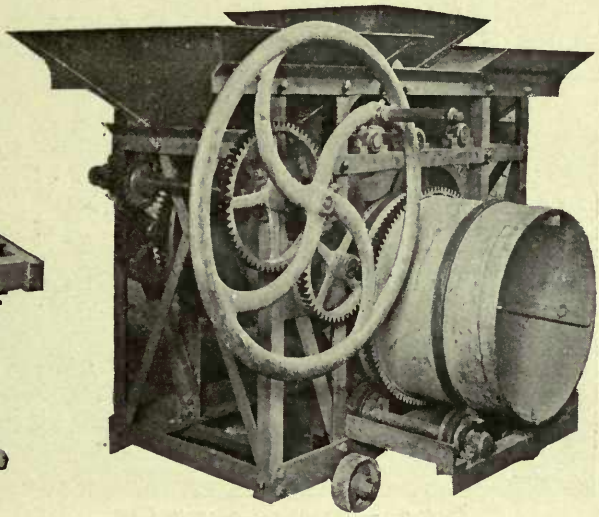


Fig. 45.—Perfect Mixer

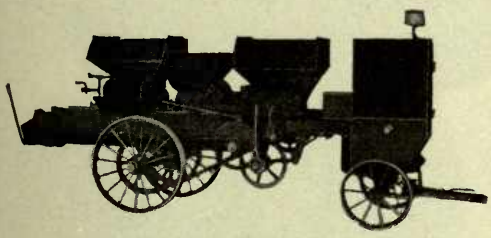


Fig. 44.—Kent Mixer

TESTING EFFICIENCY OF MIXING

Of the many methods that have been proposed for testing whether concrete has been properly mixed, the most practical and the one best suited for adoption in everyday work is that due to Dr. J. S. Owens, and described by him before the Society of Engineers. It is based on the principle of sampling a heap of concrete at different places, and then seeing whether all the

samples contain the same relative proportions of stone, sand, and cement. Each sample is put into a tall glass cylinder nearly filled with water, shaken up, and allowed to settle. The rate of settlement of cement in water is about thirty times as slow as that of the sand, which, in turn, is slower than that of the larger particles of stone; as a result of the variable rates of deposition the stones, sand and cement settle in distinct layers, whose depth is proportional to the amount of the material present in the sample. Thus it is possible to tell, with a fair degree of accuracy, whether each sample was compounded with the same proportions of cement, sand, and coarse aggregate; obviously, the test requires reasonable care in its execution.

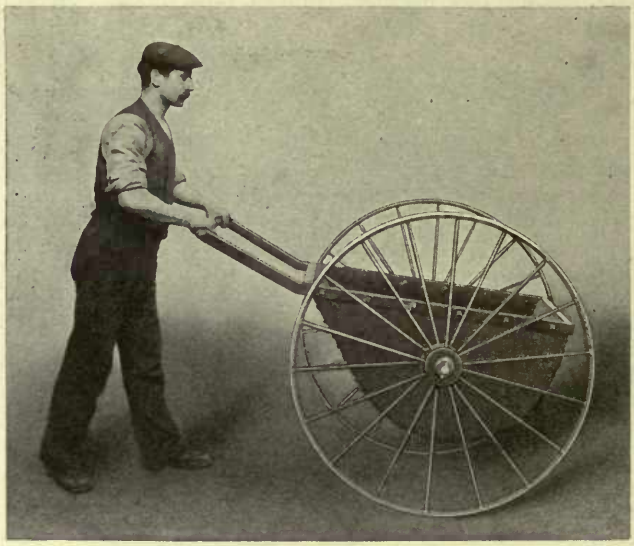


Fig. 47.—Ransome Concrete Cart

CONVEYING CONCRETE

It always happens that either the raw materials have to be conveyed from their storage place to the stage or mixer or that the concrete is made close to where the materials are stored and then has to be carried to the place where it is to be used. The means of conveyance employed embrace barrows, handcarts, derrick hoists, bucket elevators, and special hoisting systems, such as the Ransome, that have been particularly designed for use in connection with concrete buildings.

The barrows may be deep wooden barrows of the usual type, or they may have an iron framework supporting a steel

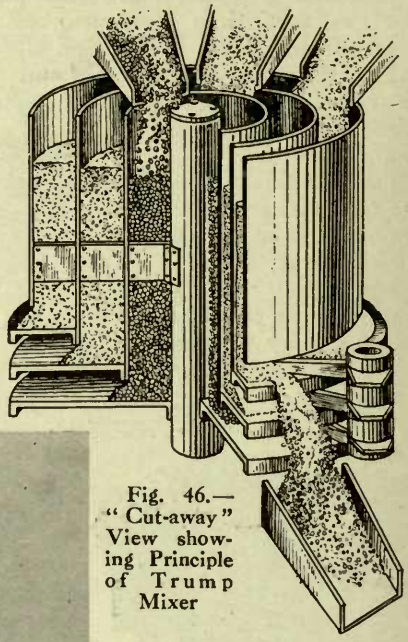


Fig. 46.—"Cut-away" View showing Principle of Trump Mixer

box in which the concrete is carried, the design being such that the load is carried as much over the wheel as possible. A convenient type of iron barrow is arranged in such a way that the box is pivoted on the axle at each side of the wheel, so that it can be tilted forward to discharge its load. In

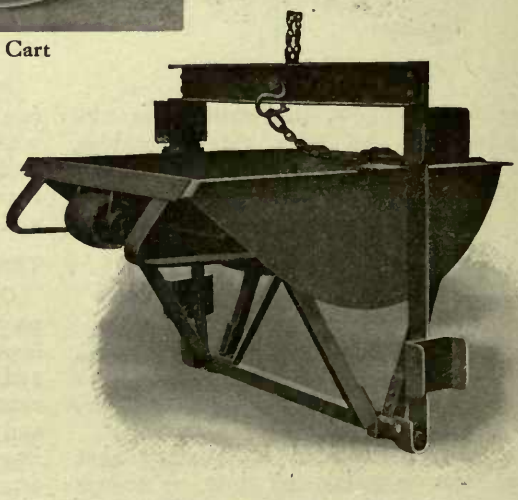


Fig. 48.—Ransome Concrete Skip or Bucket

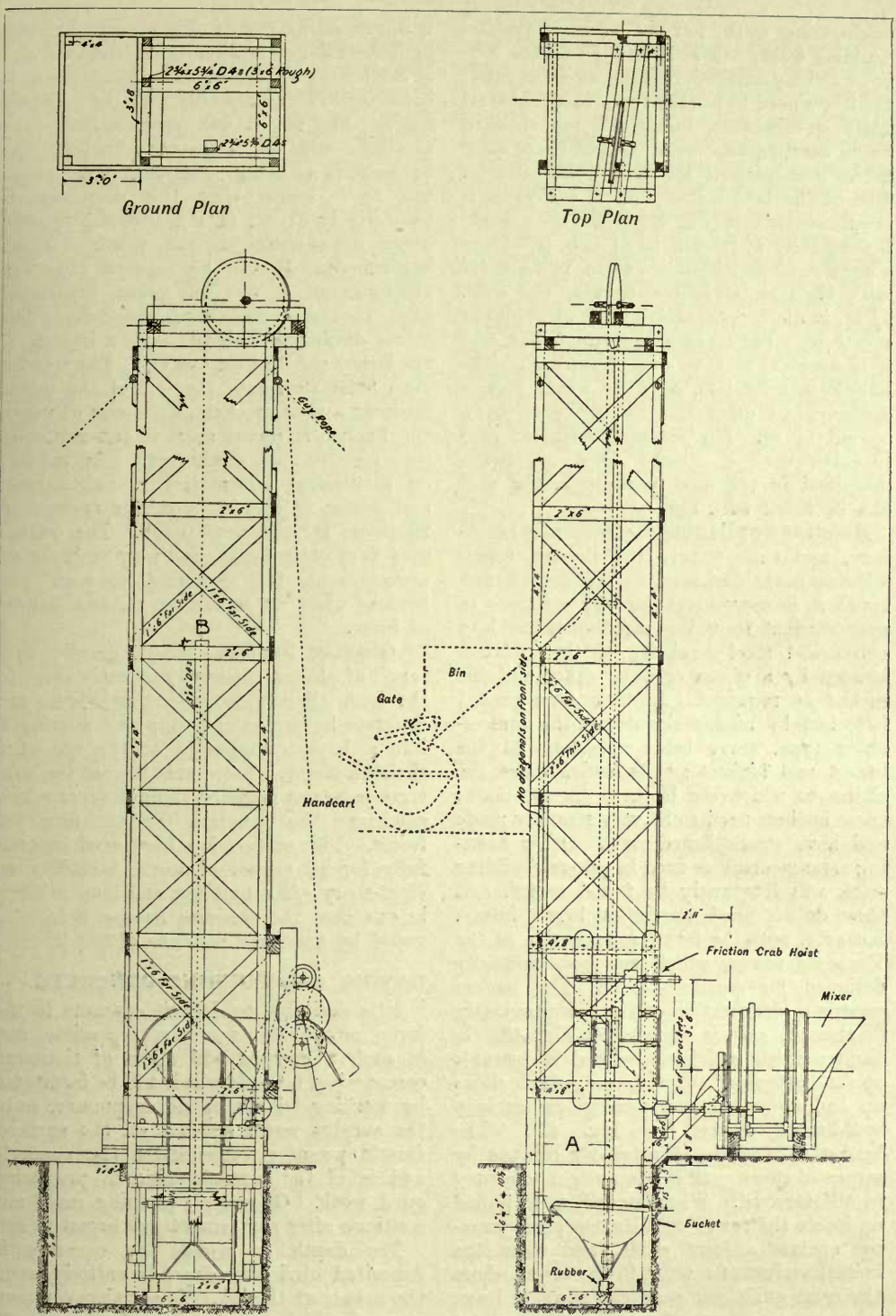


Fig. 49.—Ground Plan, Top Plan, and Two Elevations of Ransome Tower

engineering work, but not often in general building work, it is economical to have barrows or tray dumpers running on fixed rails; the boxes need to be made of substantial steel plate strengthened round the top edge to avoid bending and buckling, and there must be an arrangement for tilting the buckets. One of the best forms of hand conveyers is the Ransome cart (Fig. 47), which has a working capacity of from 5 to 5½ cub. ft. of wet concrete. The wheels are 42 in. in diameter with eighteen staggered spokes, the axle, 1¼ in. in diameter, passing right through the cart body; the tread of the tyre is flat, and it is essential to have a good runway. The handle is reversible, and the whole cart is designed to discharge its contents in a second or so. For elevator work, in conjunction with a special form of tower described in the next paragraph, the cart can be fitted with legs.

Hoisting appliances are frequently necessary, and their nature will depend somewhat upon the design of the building. Sometimes it is convenient for the concrete to pass straight from the machine mixer into cylindrical steel buckets, which are then hoisted by a steam crab or other suitable means as required. Another arrangement is to instal a bucket elevator of the endless-chain type, there being sprockets at the lowest and highest points which drive the chains to which the buckets are attached; these buckets need to be very strongly made and have strengthened rims. Other hoisting arrangements as used in general building work will frequently be found useful, and these do not need description here. Space, however, must be found for mention of the Ransome tower, which has been specially designed for concrete work. The bucket used is a modification of the one previously illustrated, and is shown by Fig. 48. It has trunnions which are carried in journals at the bottom of a steel frame, which slides up and down between wooden guides in a well-braced tower (*see* Fig. 49). The bucket is carefully adjusted for balance by means of stops until it has a slight tendency to tilt forward. When loaded, it is pulled up inside the tower with its nosepiece pressing against a front guide, and when the bucket arrives at a point in the tower where the front guide has been cut away to leave a space, as shown at B, Fig. 49, it automatically tilts forward and discharges its contents into a bin from which the concrete is drawn off by means of gates into barrows or handcarts, or in some other suitable way is carried to the work. The hoist tower is constructed of light timber, and, built according to Fig. 49, it will accommodate the smallest size of Ransome bucket, the dimension A being 30½ in., whereas in the three larger sizes this dimension is respectively 37 in., 44 in., and 50½ in., the other dimensions of the tower varying accordingly. It will be realised that the arrangement is one of great ingenuity, and, at the same time, simplicity. The mixer discharges a batch into a bucket at the lowest point of the system. The friction crab hoist shown to the left of the mixer operates a rope or chain, by means of which the bucket is raised until it automatically tilts forward and discharges. The bucket, on descending automatically, rights itself and comes to rest on a rubber cushion in readiness for the next batch. The bucket may take other forms, and may embrace an arrangement for dumping through the bottom when an attendant raises a handle or lever.

Gravity System.—The "gravity system" of placing concrete has lately come to the fore. Briefly, it consists in hoisting the concrete by means of a skip and pouring it into a hopper supported at the top of a skeleton tower. From this hopper the concrete passes as required through pipes which discharge the concrete directly into the forms. This system has been used successfully for all classes of work, including an eight-story office building and long bridges, in one case the concrete having been conveyed by gravity a distance of 502 ft.

LAYING AND POURING CONCRETE

It is desirable to deposit concrete in the forms or moulds as gently as possible, and in such a way as will allow of thorough compacting, a result which will be facilitated by working with a spade or punner, until the surplus water appears on the surface. Gentle punning consolidates the concrete and is of the utmost value in producing good work. Of course, punning must not continue after the cement has begun to set.

The depth to which the concrete is deposited obviously depends entirely upon the nature of the job. To say that common sense should be employed, and the concrete never deposited so thickly that there is doubt as to whether the reinforcement is in

contact with the concrete at every part, is better than to specify layers of any particular thickness. Much has been said as to whether concrete may properly be tipped from a height into its final resting-place, such a method having been thought at one time to be necessary in order to consolidate the material. The method is not harmful where the vertical distance or "head" is only a few feet, but it is not to be recommended when the distance is any more than that, as there is a tendency for the heavier ingredients to separate from the bulk of the concrete.

The very greatest care should naturally be exercised to prevent any interference with the wet concrete after it has been deposited and rammed. Any disturbance after setting begins is detrimental to the strength.

It is usual to cover up the concrete at night, one object of so doing being to prevent any violent rain washing some of the cement out of the top surface, and another, in winter, to prevent frost reaching the work, since it appears that frost occurring before the cement has set exercises a disintegrating action, and influences the setting power and ultimate strength unfavourably.

When depositing concrete in layers, it is important to secure a good bond between the old and the new material. When there has been any length of time between the two layers, it is necessary to wet the old work, hack it over, and sweep clean. Many authorities take the precaution of treating the first layer, after hacking and sweeping, with cement grout, so as to ensure a strong bond. When one layer follows the preceding one at a close interval, all that is necessary is to see that the surface of the concrete already deposited is wet.

Steel

Composition of Steel.—Steel consists of iron containing carbon in the form of iron carbide, the proportion of which, and the manner in which it is held, largely determining the character of the steel. Other elements are also present. Steel always contains manganese, and also small amounts of silicon, sulphur and phosphorus. The proportions of the two latter are important, and only traces not exceeding 0·04 per cent. are permitted in structural steel. The table given below indicates the relative composition of the various forms of iron in use.

	Silicon	*Carbon*		*Manganese*	*Sulphur*	*Phosphorus*
		Graphite	*Combined*			
Cast iron (castings)*	1—3·5	1—3	0·15—0·9	0—2	0—0·2	0—1·5
Wrought iron . .	tr.—0·1	—	tr.—0·25	tr.—0·1	tr.—0·1	tr.—0·2
Mild steel . . .	tr.—0·3	—	0·1—0·5	0·3—1·0	tr.—0·04†	tr.—0·04†
Harder steel . .	tr.—0·3	—	0·6—0·8	ditto	ditto	ditto
Cutlery steel . .	—	—	0·5—1·5	tr.—0·3	trace	trace

* Not pig iron or malleable cast iron.
† For rails this may be 0·06–0·07.

Alloy steels contain, in addition to carbon, other metals which give their name to the steel—nickel, chromium, tungsten, molybdenum, vanadium, and manganese—and each of these alloy steels has some special character.

It will be seen that the tough, tenacious, and malleable forms of iron are, with the exception of alloy steels, those containing only small amounts of foreign bodies other than carbon, and that as that element increases, the hardness and elasticity increase. Cutlery and tool steels contain the larger amounts of carbon.

Malleability.—No confusion should exist as to the term "malleable" applied to iron and steel. Metal that can be forged, rolled, bent and worked at a red heat, and to a less extent in the cold, is malleable. The term is often applied without qualification to certain kinds of castings which have had their brittleness removed by treatment. The composition of these varies according to the method by which they have been produced. They are cast, not forged, into shape, and the metal is not malleable in the above sense. The bending and twisting of wire used in reinforced concrete work for tying the bars or making meshed wire supports requires that the metal to be used shall possess the property of malleability.

Tenacity.—The resistance to rupture by a stretching force is known as tenacity. It is determined by subjecting a piece of the metal of known sectional area to a gradually increasing stress in a testing machine till rupture takes place. The force applied is indicated by the machine. In Great Britain, tenacity is usually stated in pounds or tons per square inch of sectional area. If a test-piece of 0·7979 in. diameter broke when subjected to a load of 30,000 lb., the tensile strength of the material would be 60,000 lb. or 26·78 tons, since the bar is half a square inch in sectional area.

The tensile strength varies with the composition, and the treatment of the metal. In carbon steels that have been heated above 900° C. and allowed to cool in the air—normalised—the tenacity increases as the percentage of carbon rises, till it attains a maximum of 61 tons at about 1·2 per cent. carbon. Steel that has been annealed by heating the metal to redness and allowing to cool very slowly has a lower tenacity. The maximum of about 36 tons is reached with 0·9 per cent. carbon, above which it falls away again. The graph

presented by Fig. 50, and the figures on the preceding page are given by Prof. Arnold.

Increasing the rapidity of cooling—by quenching from redness—may raise the tensile strength, but affects also other properties. The smaller the amount of carbon present, the less is the effect produced by varying the rate of cooling. Very soft steels, containing little carbon, are not greatly affected, as will be observed from the diagram just referred to.

The divergence in the indicated strengths between the normalised and annealed specimens becomes more marked as the percentage of carbon increases. The quenching effect also follows the same course, and is much greater with the higher carbon steels. They at the same time become much harder and more brittle. The hardening effect becomes sensible above 0·35 per cent., and with 0·45 per cent. is quite distinct. With 1 per cent. the metal becomes brittle and very hard.

Under ordinary treatment a percentage of carbon below 0·8 per cent. raises the tensile strength, but does not make the metal brittle. Steel containing from 0·7 to 0·85 of carbon, although its ductility has been greatly reduced, is quite suitable for reinforced concrete construction, as it is still sufficiently ductile to be free from liability to breaking under ordinary conditions; it is, however, much stiffer and more rigid and elastic than metal having a lower carbon content. Do steels with higher carbon contents, used in plain bars, and not ridged, notched or otherwise "deformed," secure a more permanent grip on the concrete than do steels with lower carbon contents? It is a moot point.

Ductility.—The power of being drawn out or extended is known as "ductility," and it is indicated by the extent by which the test piece in a tensile test elongates before fracture. This elongation is expressed as a percentage of the original length of the piece. Before testing, two centre-punch or other marks are made on the piece, at a carefully measured distance apart. After fracture the two portions are laid together and the distance again measured. The increase is calculated in percentage terms of the original length. In stating elongation, the length of the test-piece must be given, since, owing to the extensive thinning of the bar at the point of fracture, much of the elongation is confined to that portion in which the fracture occurs. This is the case with both short and long test-pieces, but on the short pieces the local elongation represents a higher percentage of the original length.

Ductility decreases as the carbon increases, as the graph (*see* Fig. 51 on next page) by Prof. Arnold shows. Here, again, the effect of rapid cooling is plainly indicated. Carefully annealed material shows a minimum ductility at about 0·89 per cent. carbon, after which the ductility rises. With normalised metal the decrease continues to little more than 2 per cent. at 1·47 per cent. carbon.

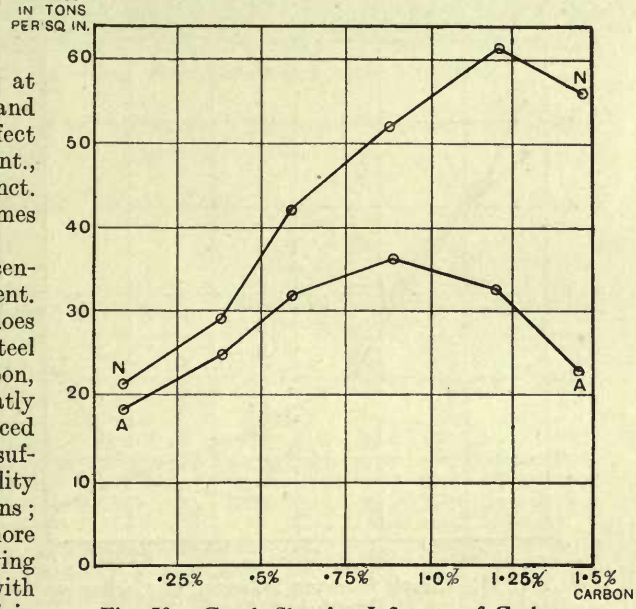


Fig. 50.—Graph Showing Influence of Carbon on Tenacity of Steel: N, normalised specimens, and A, annealed specimens

The difference in the behaviour of the metal in the two states results from the difference in the manner in which the carbide is held. The carbide dissolves and diffuses through the metal when heated. If slowly cooled it separates completely from the iron during the cooling. More rapid cooling prevents the separation from being completed, while quenching may prevent the separation altogether. Thus the properties of the metal vary with the rate of cooling and the amount of carbide.

From the above it will be seen that for material that must be subjected to severe treatment in forging and use, such as boiler plates, rivets, etc., only soft, low-carbon steels can be applied. Steel containing more carbon would be too hard and not sufficiently malleable and ductile, so that cracking or fracture might occur. With materials which are not subjected to severe treatment and where high tensile strength, elasticity, and rigidity are more important, higher proportions of carbon are allowable. Thus, with 0·9 per cent. carbon, a tensile strength of 51 tons per square inch and an elongation of 14 per cent. are obtained, the latter being somewhat low, but higher than in the case of much wrought iron, the tensile strength of which is less than that of the steel.

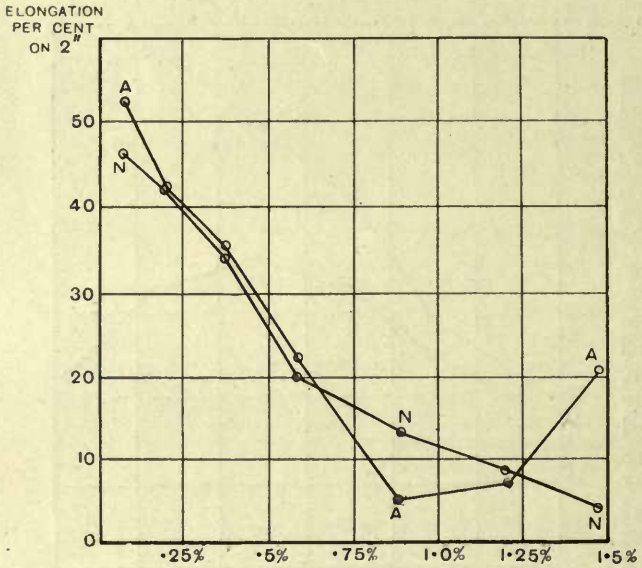


Fig. 51.—Graph Showing Influence of Carbon on Ductility of Steel

Elasticity.—This is the power of the steel to recover its original shape, and it is influenced by the manner in which the carbon is held in the metal.

Limit of Elasticity.—This term is applied to the force which will produce a permanent set or extension of the metal under tensile test; that is, the point at which the metal ceases to be perfectly elastic. It is usually about half the total strength. The metal will continue to stretch on the further application of force, but the extension is not then proportional to the force applied, and the metal does not recover its original dimensions on the removal of the stress, but is permanently strained.

Testing Steel.—Fig. 52 shows test-pieces. They are carefully machined to shape, marked by means of a centre-punch for determining elongation, and the ends firmly gripped by the jaws of the machine. Force is gradually applied to pull the ends apart and the piece carefully watched. At a certain point, a change in the outer skin may be observed. With a lever machine the sudden elongation may cause the lever to drop. This point is the commercial elastic limit. Further application of force produces further extension, and ultimately a point is reached when the increasing load causes the piece, which up to this point has been extending uniformly over its whole length, to begin to contract locally, and rapidly to diminish in sectional area; in this region the piece ultimately breaks. The contraction in area is generally stated as a percentage of the original area. The fracture on examination should be uniform, free from flaw or defect, and silky or finely granular. Round pieces should show cup and cone formation if soft. Fracture should occur near the middle, and with uniform material there should be only one region of contraction. Tests in which this does not occur or which break very near one end are not satisfactory. The effective length—that is, the distance between the marks—varies from 2 in. to 10 in.; but usually 2-in., 6-in., 8-in., or 10-in. test pieces are employed.

SPECIFICATION FOR STEEL REINFORCEMENT

The following is a suitable specification for steel reinforcement:

The steel used shall be manufactured by the Open Hearth process, and shall be of the very best description, free from blisters, scales, laminations, and shall be of the size and weight specified (a variation of 2½ per cent. either way is usually allowed).

The chemical analysis shall show — per cent. carbon, and not more than the fol-

lowing percentages of phosphorus and sulphur:

Phosphorus . .	0·06 (maximum)
Sulphur . . .	0·04 (maximum)

No allowance above these figures must be made. The steel shall have an ultimate tensile strength of not less than 28 to 32 tons per square inch, and an elongation of at least 15 per cent. on an 8-in. length. No welds shall be made in the steel under any circumstances. When forging is necessary special attention shall be paid to the smith's work. No overheated or burnt steel to be used. All bending (where possible) of small metal to be done cold. Larger metal to be heated to a dull cherry red. The bending force must be applied gradually and regularly. Before use, all steel must be clean and free from scale and rust. No oil or paint must be used on the steel.

Modulus of Elasticity.—This is the force in pounds that would be required to extend a bar to double or compress it to half its original length on the assumption that it remained perfectly elastic throughout. It is frequently called the coefficient of elasticity; thus for steel the modulus is about 30,000,000 lb., and the coefficient $\frac{1}{30,000,000}$. It is fairly constant for open-hearth steel with varying carbon contents.

On this assumption a force of 1 lb. would extend or reduce the length of a test-piece by $\frac{1}{30,000,000}$ of the original.

CERTIFICATE OF TESTS

The following is a sample certificate of tests for steel used in reinforced concrete construction:—

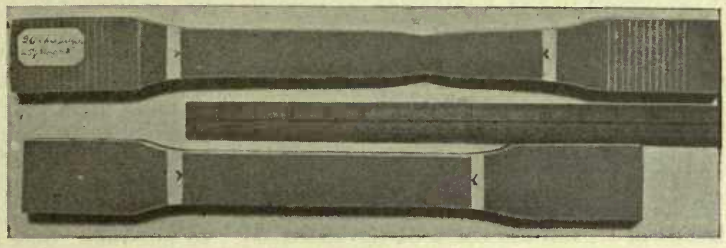


Fig. 52.—Test Piece Before and After Stretching

CERTIFICATE OF TEST

Messrs. X.
Tests of Mild Steel to your order.
For delivery to A. B.

No. of Sample	Original size		Contraction of Area			Elongation on 8 inches		Ultimate stress		Elastic limit	
	Dimensions	Area	Dimensions	Area sq. inches	Per cent. contraction	Final length in inches	Per cent.	Actual tons	Tons per sq. inch	Actual tons	Tons per sq. inch
1⅛ Rounds .	1·12	0·985	0·76 Diam.	0·454	54	10·22	27·7	28·6	29	18·64	18·9
1 × ½ Flats .	1·01 × 0·495	0·5	0·79 × 0·295	0·232	53·5	10·24	28·0	14·6	29·2	9·75	19·5
Ditto . .	1·00 × 0·505	0·505	0·75 × 29	0·217	57·0	10·16	27·0	14·4	28·5	10·10	19·9
1 inch square	1·00 × 0·99	0·99	0·7 × 0·69	0·482	51·3	10·25	28·2	29·8	29·9	20·8	21

1 × ¼ Bend tests satisfactory.
Cold Bend tests satisfactory.
1 × ¼ Flats cold bend tests satisfactory.

Fatigue.—When subject to varying and fluctuating stresses, metals are liable to alteration. The tensile strength, ductility, and toughness are reduced. In some cases this is very pronounced, and the strength may fall below the ordinary working load and fracture take place. The alteration under ordinary conditions takes place slowly in steels that are free from injurious constituents and of uniform texture. The elasticity generally depends on the latter condition, and metals of high elasticity are less liable to fatigue. The elastic limit in the case of steel is higher than for wrought iron, and its resistance to internal change is greater.

Toughness.—This may be defined as the resistance to fracture by bending beyond

the elastic limit. It is especially necessary in steel that must be bent or twisted. Soft malleable steel, of uniform texture and character, is toughest; but very much depends on the heat treatment the steel has received.

Hardness.—This is difficult to determine with accuracy, and for reinforced concrete work is only of secondary importance. It is dependent on the heat treatment and rate of cooling of the metal, and on the percentage of carbon present in the steel. Reference to the following table, and a consideration of the uses of steels of varying composition, will illustrate the latter point, and the study of the effects of carbon to be dealt with later will explain the cause of the increased hardness produced by quenching from a red heat.

Steel	*Carbon contents*
Soft malleable metal for plates and rivets . .	0·1 —0·3 0·2 —0·4 Ma.
For ordinary wire for ropes	0·15—0·3
Hard wire for guide ropes .	0·45—0·75
Rails	0·3 —0·6
Rolled sections . .	0·22—0·25
Hard open-hearth . .	0·6 —0·8
Tool steel for dies and steeling purposes (axes and plane irons, die temper) .	0·75
Setts, minting dies, smith's tools (sett temper) .	0·825
Cold chisels, miners' drills, large punches (chisel temper) . . .	1·00
Circular cutters, taps, rimers, large turning tools and drills, screwing dies (punch temper) .	1·125
Turning tools . . .	1·25
For small tools, sawfiles, etc.	1·375
For razors and special purposes . . .	1·5

High-speed steels containing tungsten, chromium, and other metals to render the metal self-hardening contain only 0·5—0·7 per cent. carbon.

Referring to the specification already given, freedom from blister may be secured by using ingots free from blow holes. Scales and laminations result from faulty treatment in the forging or rolling, or to some foreign matter included in the steel. Overworking the steel may produce lamination.

Usually with increased diameter a smaller demand is made for elongation. For an increase of $\frac{1}{8}$ in. above $\frac{3}{4}$ in. a deduction of 1 per cent. is made, for decreases of $\frac{1}{16}$ in. below $\frac{5}{16}$ a deduction of $2\frac{1}{2}$ per cent. from the specification.

Bending Tests.—These consist of bending the metal as per specification upon itself. Rivet steel is tested at full size as rolled. It must bend flat on itself—through 180°—without fracture on the outside of the bend. All material less than $\frac{3}{4}$ in. is tested at full thickness, but above that thickness the specimen used is $\frac{1}{2}$ in. thick, and if possible $1\frac{1}{2}$ in. wide. Soft steel must bend over on itself—through 180°—without fracture at any part of the bend. Medium steel must bend through 180°, round a diameter equal to the thickness of the metal, so that the two limbs are separated by a distance equal to the thickness of the test, without showing sign of fracture on either side. For works tests, the pieces are often doubled closely together. Tests after quenching from a red heat are somewhat less severe, the distance between the limbs being equal to $1\frac{1}{2}$ times the thickness of the metal.

The importance of a high elastic limit will be appreciated when the effect of subjecting to tensile strain a smooth round bar, one end of which is embedded in concrete, is considered. In such circumstances the load may be increased till the elastic limit is reached without in any way permanently lessening the grip of the concrete on the metal. When the elastic limit is

TENSILE STRENGTH AND ELASTIC LIMIT OF STEEL IN TENSION AND COMPRESSION IN POUNDS PER SQUARE INCH.*

Percentage of Carbon	*Ultimate tensile strength*	*Elastic limit in tension*	*Elastic limit in compression*
0·14	62,944	41,955	39,536
0·19	68,096	47,084	43,008
0·46	75,712	49,056	48,944
0·55	80,416	46,995	49,772
0·66	89,600	53,244	53,648
0·78	92,064	53,312	53,648
0·82	102,816	57,008	63,168
0·87	104,608	61,017	56,000
0·96	118,048	69,216	71,120

* Quoted from Twelvetrees' "Concrete-Steel."

reached, the permanent extension with the accompanying thinning of the bar would release it as far as contraction extends, and this would probably progress along the bar by the repetition of the stress.

Shearing Strength.—The value of this factor is very largely influenced by the structure of the steel developed in rolling and the heat treatment. Usually it amounts to 70 to 75 per cent. of the ultimate tensile strength.

MODULI OF ELASTICITY OF STEEL WITH DIFFERENT PERCENTAGES OF CARBON IN 1,000,000-LB. UNITS.

Carbon	*Tension*	*Compression*	*Bending*	*Mean*
0·19	30·8	37·0	29·1	32·9
0·46	32·0	32·7	29·3	31·8
0·54	30·6	36·1	28·8	32·4
0·66	32·4	35·7	32·1	33·6
0·78	33·5	32·4	30·1	32·4
0·87	31·0	31·5	30·4	31·1
0·96	30·9	32·7	29·2	31·2

The effect of subjecting bars to mechanical treatment, and the consequent variation in the results obtained, is evidenced in the table given below. The bars tested had been twisted at one end while cold.

Resistance to Alternating and Repeated Stresses.—For all structural purposes the permanence of the steel is of the highest importance. Repeated stresses induce a brittle condition which is not indicated by the testing machine. The piece breaks as though it were absolutely brittle. Tests of brittleness are made by bending small prepared test-pieces in opposite directions through a small angle by means of a suitable machine. The number of bendings before fracture occurs varies with the composition and the treatment the steel has received. Mr. Stead gives the following figures for steel containing 0·44 per cent. carbon:

Normal . . .	1,432,500
Overheated . .	844,950
Reheated . . .	2,080,440
Annealed . . .	1,971,000
Sorbitic . . .	3,517,200

The rate of alternation has a very considerable effect. Prof. Arnold has shown that the resistance to rupture is inversely proportional to the rate of alternation. Steel that has become brittle under such conditions cannot be restored by annealing, neither can it, indeed, by any other heat treatment.

Description of Bar	*Original size in inches*	*Original area in sq. inches*	*Elastic stress per sq. inch*	*Ultimate strength per sq. inch*	*Ratio of Elastic to Ultimate*	*Contraction in area at fracture*	*Extension in 8 inches*	*Extension in 10 inches*	
5·88 *twists per foot*									
Plain end .	·377 ×	0·144	43,000 lb. =19·2 tons	58,847 lb. =26·3 tons	73·1	71·5	27·5	26·2	Silky
⅜ square .	·382								
Twisted end	do.	do.	59,700 lb. =26·7 tons	86,250 lb. =38·5 tons	69·2	59·7	6·7	6·0	do.
4·21 *twists per foot*									
Plain end .	0·5 ×	0·25	44,000 lb. =19·6 tons	62,400 lb. =27·9 tons	70·5	67·6	28·7	27·6	do.
½ square .	0·5								
Twisted end	do.	do.	78,400 lb. =35·0 tons	88,480 lb. =39·5 tons	88·6	56·4	7·2	6·3	do.
3·15 *twists per foot*									
Plain end .	0·613 ×	0·377	31,500 lb. =14·1 tons	53,475 lb. =23·9 tons	58·9	69·8	35·5	33·9	do.
⅝ bar . .	0·615								
Twisted end	do.	do.	64,000 lb. =28·6 tons	73,820 lb. =33·0 tons	86·7	63·7	7·0	6·0	do.

Effects of Heat on Physical Properties.—Sir W. Fairbairn showed that the strength of rivet iron and boiler plates (best Yorkshire iron) increased as the temperature rose to 435° F., rising from 62,720 lb. at 60° F. to 86,016 lb. at 435°. At 1290° F. the strength was only 35,840 lb. Mild steel does not so increase in strength. Styffe showed that between 212° F. and 392° F. the strength remained practically unchanged.

Other experimenters, including Howard, Barnaby, and Martens, have shown that between 60° F. and 150°—250° there is a fall in tenacity, followed by an increase up to 450° or 600° F. Beyond this point the tenacity falls, and at 1100° to 1200° F. is only about half the original.

Although an actual increase in strength may be registered, the ductility is diminished. At about 550° to 570° F. the metal shows signs of brittleness, and fracture occurs suddenly without elongation. Resistance to shock at or above these temperatures is therefore probably much diminished. This temperature, be it noted, is much below redness.

The minimum length of a bar to be embedded in concrete in order to ensure its not slipping under tension will depend on the adhesion between the two substances, the elasticity and the area of the metal surface embedded. Until the bar stretches the adhesion will remain unimpaired. It may, however, be affected by unequal expansion of the materials, when the composite material is subjected to heat and also to internal stress of a variable character such as vibration and shock. Any advantages derived from the shape of the bar is due either to the increased area caused by the shape given or to the irregularity of form, securing a bond when the natural one may have been reduced or destroyed.

The adhesion varies greatly, and is influenced by the nature of the cement, the amount of water used and other factors.

Coefficient of Expansion.—The coefficient of expansion of steel is 0·000066 for mild steel, and 0·000069 for hard steel.

Other Elements Present in Steel.—Beside carbon, steel also contains small amounts of other elements.

Silicon.—Usually only traces of this element are present, being the remains of what was present in the pig iron used in making the steel, and which was not eliminated during the process. It is generally below 0·1 per cent. In such amounts its effects are negligible. Small amounts of silicon in steel have the effect of ensuring sounder metal freer from blowholes, and it is often added for this purpose to the metal before casting. In larger amounts, silicon has the effect of largely increasing the size of the crystal grains, and with 2·5 per cent. the metal presents on fracture an appearance more or less like spiegel. Further increase produces a fracture resembling silicon iron. Turner found that up to 0·315 silicon had no effect on the tensile strength or ductility. With 0·5 per cent. an increase of 7 tons in the tensile strength and a diminution of 4 per cent. on the elongation took place.

Larger amounts of silicon have the effect of raising both the tensile strength and the elastic limit of the metal. With small proportions of silicon, no loss of ductility occurs, but with amounts beyond 2 per cent. the loss in ductility is very marked, and with 4 per cent. the ductility is practically nil. The metal remains malleable if the carbon content is low, until more than 6 per cent. of the element is present. Beyond this it is brittle. With increased carbon less silicon will produce brittleness. Silicon up to 2 per cent. has no effect on the hardening of steel by quenching in water even from a welding heat. More than this renders the steel somewhat stiffer, but not harder. Silicon steels are being used for tram rails and in work where high elasticity is required. It has been applied to the making of springs for motor-cars and other vehicles For rails 0·3 per cent. with 0·5—0·6 per cent. carbon and 1 per cent. manganese, and for the latter 0·8—0·9 per cent. silicon and 0·7 per cent. carbon and 0·4 per cent. manganese. With the latter alloy, very careful treatment is necessary to develop the mechanical properties. Carefully annealed, the tensile strength is about 50 tons, with an elastic limit of 30 tons, and 14 to 18 per cent. elongation. After quenching at about 900° C. and reheating to 500° C. the tensile strength is 85—95 tons, elastic limit 63—67 tons, with an elongation of 5 to 12 per cent. The table on p. 57 is quoted from Greenwood and Sexton's "Steel," and shows the chief mechanical properties of the metal.

Manganese is a constant constituent of mild steel. Iron free from or containing little carbon cannot be melted and cast without becoming burnt. Manganese is

Analysis		*Unannealed*				*Annealed*			
Per cent.		*Tons per sq. in.*		*Per cent.*		*Tons per sq. in.*		*Per cent.*	
C.	*Si.*	*T.S.*	*E.L.*	*E.*	*C.*	*T.S.*	*E.L.*	*E.*	*C.*
0·14	0·24	33	22	30	54	25	15	37	61
0·18	0·73	34	25	29·5	54·5	29·5	19	34	52·5
0·19	1·6	37·5	28	31	50	33	25	35	54·5
0·20	2·18	39·5	31	18·5	28	34	25·5	36·5	60
0·20	2·7	42·5	32	17·5	24	32	24	6	6·5
0·21	3·4	47·5	35	11	14	39	30	9	9
0·25	4·2	49	45	0·004	0·2	38	none	0·64	0·98
0 26	5·5	48	none	0·3	0·7	25	25	0·37	0·5

added in the form of spiegeleisen or ferro-manganese to deoxidise it and thus restore malleability. Its tendency is to make the metal harder and to raise the tensile strength slightly, but to reduce the ductility. It may perhaps correct to some extent the effect of sulphur in producing red shortness. For structural steel the amounts present vary from 0·2 to 1 per cent. The latter is allowed in rails and is occasionally exceeded. Generally the amount is below 0·6 per cent.

Phosphorus.—This element is present in steel as the phosphide, containing 15·57 per cent. of phosphorus ; that is, phosphorus produces nearly 6½ times its weight of the compound, and the effects must be judged in this light. Its general effect is to produce cold shortness, brittleness, and sensitiveness to shock. It produces a crystalline structure. The metal rolls well hot, and its tensile strength and elasticity are raised, but the ductility is reduced, and may be completely destroyed. The effect on the ductility and toughness is intensified by the presence of carbon. With amounts under 0·1 per cent. but little change can be observed in the metal under test, but its sensitiveness to shock may be very greatly increased. Although its behaviour varies, its reliability in this respect is destroyed. It is necessary therefore to specify a maximum. For rails and similar purposes a maximum of 0·08 is permissible, and 0·07 is usually specified, but 0·04 is often required in steels containing 0·4 to 0·6 carbon. With higher-carbon steels less phosphorus is allowable, and the lower figure should be adhered to. For tool steels the phosphorus should not exceed 0·02 per cent.

Sulphur.—This element also exercises, even in small quantities, a serious influence on the properties of steel. It induces red shortness when the amounts present are not sufficient to produce any effect on the tenacity or ductility of the steel in the cold. The steel cracks on rolling even when the sulphur present is less than 0·1 per cent., and the steel is therefore very unreliable, since, although externally satisfactory, internal cracks and flaws may be produced which, under the conditions of application —vibration, variable temperature, etc.— may ultimately weaken the steel below its working load and produce fracture. It is most likely to produce fatigue, and its presence destroys the welding power of the steel. Not more than 0·05 per cent. should be allowed in any circumstances.

Carbon.—As already noted most of the changes in the mechanical characters of steel are produced by the carbon present. Not only does an increase in the element produce a change in properties, but different properties may be presented by steels containing the same amount, if they have received different heat treatment. Steel heated to redness and quenched is made harder to an extent depending on the amount of carbon present. The same steel, if reheated and allowed to cool slowly, is softened. In the softened state it is harder than iron free from carbon. The tensile strength is raised, but the ductility is lowered. The amount is not, however, in proportion to the increase of strength. This renders carbon the best hardening agent, as it produces the greatest increase in tenacity with the least diminution of ductility. In hardening, the change occurs at a definite temperature or at least over a very limited range of temperature. If the steel cools below this temperature before quenching, it is not hardened, nor is it softened unless this temperature is reached.

Below this temperature it is immaterial whether the cooling is rapid or slow. The temperature is known as the "critical temperature" of hardening, and also as

Fig. 53.—Photo-micrograph of Hard Steel (3 per cent. carbon) Cooled Slowly

the "recalescence point." Steel of suitable carbon content and volume actually becomes visibly hotter as it cools from a red heat when the critical temperature is reached. Hence the term "recalescence." It marks the temperature at which the carbon contents of the steel change their condition, this coinciding with the hardening effect.

Carbon exists in steel in chemical combination with iron. The carbide that has been separated has the composition Fe_3C, and is insoluble in iron below the recalescence point, but soluble above it. If quenched from a higher temperature its separation is prevented, and it remains in solid solution, in which condition it produces its maximum effect. The condition it assumes after or during separation may seriously affect the properties. A microscopic examination of polished and etched steel shows that when cooled in air the carbide has collected into small areas, surrounded by uncarburised iron. The size of these areas increases with the carbon contents, and with 0·9 per cent. carbon the whole surface is uniform. The grey areas are seen under a high magnification to consist of alternate plates of light and dark material. One of the components is the carbide—Fe_3C (cementite), and the other the iron (ferrite) from which it has separated. If cooled very slowly the structure is coarsely laminated. Cooled more quickly it is finer, and the laminated structure may be very fine or even granular. The latter condition, which is stronger and more elastic, is the sorbitic structure. The laminated material under inclined illumination often shows a play of colour and was named by its discoverer, Dr. Sorby, pearlite. Heated beyond the recalescence point the cementite first dissolves in the iron (ferrite) of the pearlite, and then diffuses through the mass. On quenching, its condition remains what it was in the heated state, and the hardening effect is produced.

If cooled slowly, steels containing more than 0·89 carbon first separate the surplus cementite, and this continues till the recalescence point is reached. The structure is shown at Fig. 53. If less than 0·89 per cent. carbon is present, the excess of iron (ferrite) first separates, and the carbide concentrates into what ultimately become the pearlite areas (*see* Fig. 54), final separation and lamination being produced at the recalescence point. The more complete the separation, the softer the material will become, and the less will be the effect of the

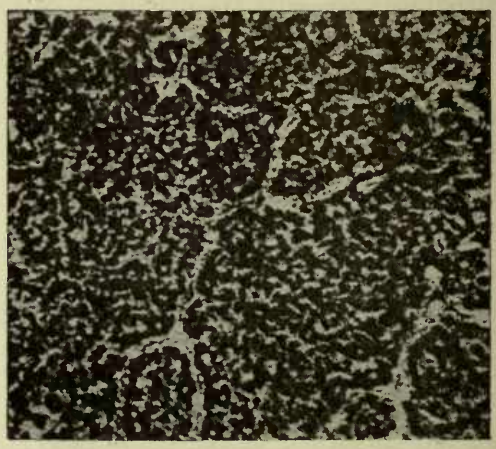


Fig. 54.—Photo-micrograph of Steel (Dark Areas show Pearlite)

carbon in raising the tensile strength and elasticity. With largely laminated pearlite the ductility is not increased, as must always be the effect of producing planes that may become planes of cleavage or separation.

Obviously the qualities of quenched steel

will depend on the carbon (carbide) present in solid solution, and at the recalescence point it is only capable of retaining carbide equal to 0·89 per cent. carbon. Above that temperature it will retain more in solution, but this, as shown, separates as the temperature falls towards the critical point. The solid solution of cementite in quenched steel is known as martensite. It is harder, stronger, more elastic, and less ductile in proportion to the carbon it contains. For structural purposes the modification of the cooling rate to prevent lamination of the pearlite by somewhat hastening the cooling, so as to produce the sorbitic structure, is sometimes carried out.

Fig. 55.—Photo-micrograph of Laminated Pearlite in Steel after Prolonged Annealing

Prolonged annealing and slow cooling will produce the coarsest lamination and the weakest metal.

Normal air-cooled metal holds a position intermediate between the two.

With regard to the photo-micrographs here shown, Fig. 53 shows hard steel (3 per cent. carbon) cooled slowly; the borders of cementite (Fe_3C) surround the pearlite areas. Fig. 54 is a low magnification in which the dark areas represent pearlite. Fig. 55 (a very high magnification) shows laminated pearlite in steel after prolonged annealing.

In carbon steels the recalescence temperature varies from 655° C. to 680° C. and coincides with the temperature at which the carbide areas assume the pearlitic structure. Visible redness is about 580° C. In hardened steels the metal is in a condition of internal stress, due to the retention of the carbide in a condition that is not normal in the cold. The raising of the temperature in tempering operations gives a certain amount of molecular liberty, and to some extent the martensitic condition is destroyed, the tendency being for the carbide to assume the normal pearlitic condition. Softening takes place to some extent, and the brittleness is reduced. The nearer the temperature approaches the critical point and the greater the latitude given, the greater is the change. At the critical point it is completed.

With mild steels there are other temperatures at which the rate of cooling is retarded. The highest point is at 880° C. Another occurs at 735°, and the third about 680°. The latter coincides with the critical point already noticed. In steels free or nearly free from carbon this is insignificant. As the carbon content increases, the upper retardations diminish and finally disappear, and the whole retardation is concentrated at the lower point. The higher critical points are ascribed to molecular changes in the iron itself. These cannot, however, be made permanent by quenching, and in the absence of carbon their effect is not noticeable in the metal, whether cooled rapidly or slowly.

In alloy steels such as tungsten, chrome, chrome - tungsten, molybdenum, nickel, manganese, etc., the added metals lower the recalescence point and bring it below ordinary temperature. Such metals in consequence retain their hardness and are known as self-hardening or air-hardening steels. They can be heated without becoming softened, and hence cutting speeds which develop so much heat as to destroy the temper of ordinary steel tools can be employed without risk. Hence the term rapid-cutting, high-duty, or high-speed steels.

METHODS OF MANUFACTURE

Blister or cementation steel is manufactured by heating bars of nearly pure iron embedded in carbon for a long period. Such metal is covered with blisters, hence the name. It is usually converted into shear steel by welding together a number of such bars to secure greater uniformity.

Cast crucible steel is made by melting blister steel in crucibles and casting into ingots. Different tempers are secured by additions of pure soft iron, carbon, and other ingredients to obtain the desired product. The principal use of such steel is for cutting

instruments. The process is costly and the output limited.

Bessemer steel is made by blowing air through molten pig iron contained in a suitable vessel. The metal is introduced in the molten state. The carbon, manganese, and silicon in the metal are removed by oxidation, and sufficient carbon is introduced by adding the necessary amount of a manganiferous pig iron—spiegeleisen—to the metal. The operation is very rapid, a charge of several tons being converted in from 16 to 20 minutes. The metal is cast into ingots and rolled. In the *basic bessemer* process, the lining of the converter is made of basic material—calcined dolomite—and this permits of the removal of the phosphorus. The operation is somewhat longer. In both of the processes, exact control of the carbon contents is difficult. Owing to the rapidity with which the operation must be conducted, there is no time for its determination, and it is impossible to hold the metal in a sufficiently fluid state for the necessary length of time to enable this to be done, as all the heat required is generated in the vessel by the oxidation going on during the passage of the air through the metal.

Open hearth steel—usually specified for reinforced concrete work—is made from pig iron and steel scrap in a gas-fired furnace of special design, known as a regenerative or open hearth gas-fired furnace. In this furnace the materials are melted, and the carbon and other elements removed by the action of air and by adding iron oxide to the charge. The heat required to melt and keep the metal molten is supplied by the burning gas. The time occupied extends over some hours, dependent on the weight of the charge and other conditions. Facilities are thus afforded for the careful control of the contents of carbon and other elements, and metal of a definite composition can be obtained. In the *basic open hearth*, a basic lining is used, and, as in the basic bessemer process, pig-iron containing phosporus may be used, as that element is eliminated during the process.

Basic steel is the term applied to steel made in converters or furnaces lined with basic materials, such as calcined dolomite, magnesite, and chromite.

Acid steel is steel made in converters or furnaces lined with siliceous materials, such as silica sand and bricks, or ganister.

Stress Simply Explained

EXPERIENCE shows that, in a large proportion of cases, the failure of the practical man, and even of the student, to grasp the essential ideas underlying the theory of construction in reinforced concrete is a direct result of his ignorance of the simple mechanics of construction, particularly his lack of acquaintance with " stress "—a term meaning the effect produced on a beam by loading it. The present intention, therefore, is first to deal with the fundamental principles that must be understood by anyone desirous of successfully studying the constructional design of buildings, and then, in the succeeding chapter, to show how those principles are applied in reinforced concrete construction.

It must here be emphasised that it is absolutely essential that the reasoning upon which the various formulæ are based should be understood if these formulæ are to be used intelligently. Students frequently fail to grasp the theory of design owing to the fact that they do not start with a sufficient knowledge of the principles of force and resistance. In many instances the very expressions that are used are imperfectly understood, and the students are endeavouring to obtain the value for something of which they do not know the meaning, and often when they have found a result they do not realise why it was necessary to find it. To the general reader this may appear to be an exaggeration of the state of affairs, but we are merely expressing views that have been gathered during several years of tuition to building students of all kinds.

FORMULÆ

There are two kinds of formulæ which are met with, namely " rational " and " empirical."

" Rational " applies to formulæ that have been deduced by reasoning alone, such reasoning being based upon definite axioms known to be correct. It is not sufficient, therefore, for the student to know a rational formula ; he should also know and understand the construction or reasoning through which the formula was evolved.

" Empirical " formulæ are based upon actual experiments through which certain values have been obtained, according to the material, load, and disposition under the test ; and, as such, their application is usually limited, and not possible for cases which do not agree with the conditions of the test.

Again, it will be necessary to use certain factors or constants derived by the person responsible for the test, and these constants must be accepted as being correct, and become merely a matter of memory and not of reasoning. There are some cases, however, where pure theory is liable to give slightly inaccurate results, owing to the peculiar behaviour of some materials under a varying stress, and any inconsistency of this kind would result in the theory not agreeing with the practice. An excellent instance of the difference between theory and practice is that explained in connection with columns and struts on a subsequent page. Wherever possible, rational formulæ are to be preferred, and it is with these that the following notes will deal in the first instance.

PRINCIPLE OF MOMENTS

By far the most important of all the points to be grasped is that of the " principle of moments," and the definition of this principle is often given in flowing language and repeated by the student without a proper understanding.

A " moment " really means an amount, and is a distinctive term, inasmuch as it means not only the actual amount of a force in pounds, hundredweights, or tons, but the actual value of a force to cause stress at any point, this value being dependent on its amount, direction, and disposition in relation to other forces or constructional members. Thus, " taking moments round a certain point " really means " taking values or amounts acting at that point."

Now, the " principle of moments " is that when several forces are acting at a point and equilibrium is produced, then all those forces that would tend to cause movement in one direction are exactly equalled by all

those forces that would tend to cause movement in another direction.

This principle may seem somewhat obvious to many readers, and they may not realise its value; but upon consideration it will be seen that it is an axiom which is frequently used. For example, in the case of a beam that carries a load at any point, it is obvious that the load is a force tending to cause movement in a downward direction, and if equilibrium is maintained, then there must be some force or forces at work which are causing or exerting an upward tendency; this force is supplied by the reaction or reactions, which together must equal the load.

LEVERS

Before investigating the method of ascertaining the value of the reactions with any given load, it will be advisable to give a few notes on the three orders of the levers. If these levers are thoroughly understood, matters will be greatly simplified, as most problems, both in ordinary construction and in reinforced concrete work, can be practically reduced to a question of leverage. A lever can be defined as a member which turns on a point known as the fulcrum, and it is the relative positions of the forces acting on the lever, and the fulcrum, which give us the order of the lever.

"**First Order.**"—A common example of a lever is that illustrated in Fig. 56, which shows a crowbar being utilised to lift up a weight, such as a block of stone. It will be seen that the lever is tending to turn on the fulcrum under the action of the two forces which oppose one another through the medium of the lever; this is an example of a lever of the first order, as the fulcrum is between the weight and the power. Now the value of the power and the weight to cause the turning of the lever will depend not only upon their actual amount in pounds or hundredweights, but also upon their distances from the fulcrum. In other words, the "moment" of the power at the fulcrum is equal to its amount in pounds multiplied by its distance to the fulcrum, the latter being known as the long arm of the lever; this is, therefore, 20 lb. × 30 in. = 600 in.-lb. The weight is opposing the power, and if equilibrium is produced, then the "moment" of the weight at the fulcrum must also equal 600 in.-lb., and this moment is found by multiplying the amount of the weight by its distance to the fulcrum. Therefore weight × 6 in. = 600 in.-lb.

$$\text{Weight} = \frac{600}{6} = 100 \text{ lb.}$$

To produce equilibrium, then, we must have

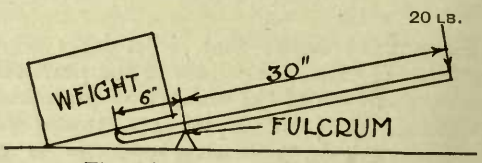


Fig. 56.—Lever of First Order

a weight of 100 lb. and a power of 20 lb. if the arms of the lever are 6 in. and 30 in. respectively. Any fraction over 20 lb. in power would raise the weight of 100 lb., and thus there is a mechanical advantage or a gain in the value of the power by the introduction of the lever. A rule can now be deduced which will apply to all levers and enable us to ascertain the weight or the power required to produce equilibrium if the value of one force is known, as follows: The power multiplied by its distance to the fulcrum is equal to the weight multiplied by its distance to the fulcrum.

"**Second Order.**"—In the first order it was seen that the fulcrum was situated between the power and the weight, while in Fig. 57 an illustration of the second order is given, where it will be seen that the weight

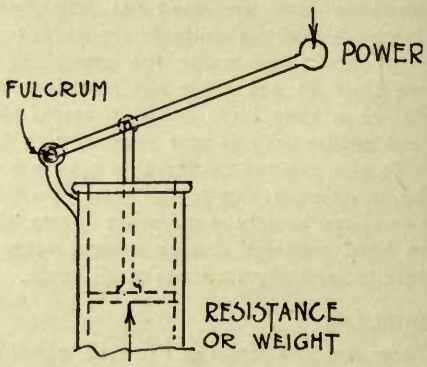


Fig. 57.—Lever of Second Order

is situated between the fulcrum and the power.

"**Third Order.**"—In this (Fig. 58) the power is between the weight and the fulcrum. The power required will always be in excess of the weight, which is not the case with the other orders. The student is advised to work out examples in each order by the rule

given above, in order to familiarise himself with the principle of taking moments round the fulcrum.

REACTIONS

A reaction, as its name implies, is a force

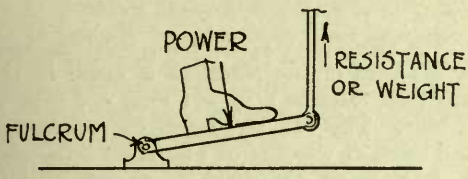


Fig. 58.—Lever of Third Order

or resistance which is the outcome of an action or force. It is obvious that, when any force or action is introduced, and equilibrium is still maintained, there must be a reaction or reactions which are equal to the initial action, but opposite in tendency. No difficulty should present itself in ascertaining the values of these reactions, but the student should fully understand the method of calculation under any condition of loading before seeking to understand the theory of bending moments.

The reactions in the case of a beam can be defined as the passive resistance at the supports offered by the strength of the materials at these points to resist crushing.

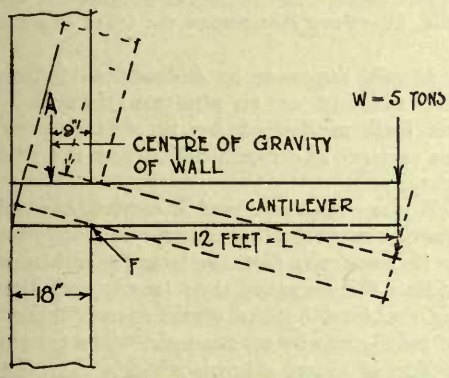


Fig. 59.—Diagram showing how a Cantilever Tends to Move

The resistance or reaction at each support will depend on the amount of the loading and its disposition on the beam.

Cantilever Reactions.—In the case of a cantilever, as there is only one support, the whole of the reaction must be supplied by this; at the same time there is a tendency for the cantilever to overturn, due to the leverage of the weight, and lift up the work above the tailing-down portion. To prevent this overturning movement from occurring, a *downward* reaction or resistance will be required at the support, and its value must be equal to or greater than the moment of the load. This resistance will be supplied by the weight of the brickwork or stonework over the tail end of the cantilever; or, if this is insufficient, anchor bolts may be carried down into the lower part of the wall.

In Fig. 59 the tendency to move is shown diagrammatically by the dotted lines, and the problem will be quite simple if the arrangement is considered as that of a lever of the first order. The overturning moment at the point F, which is the fulcrum, will be equal to the weight multiplied by its distance to this point, which equals 5 tons ×

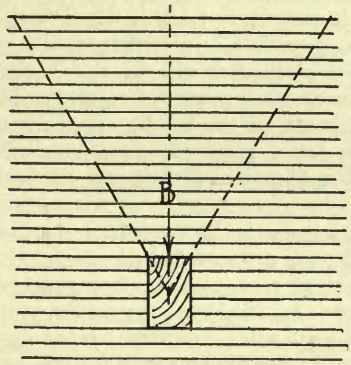


Fig. 60.—Triangular Portion of Wall Lifted by Movement of Cantilever

12 ft. = 60 ft.-tons. The weight of the wall above is distributed over the length of 18 in., and for the purpose of calculating its moment at F, it may be considered as acting at its centre of gravity, namely, 9 in. from the fulcrum. The weight of the wall acting at A multiplied by 9 in. or $\frac{3}{4}$ ft., must therefore equal 60 ft.-tons = expressed as an equation:

$$A \times \tfrac{3}{4}\text{ ft.} = W \times l$$
$$A \times \tfrac{3}{4}\text{ ft.} = 5\text{ tons} \times 12\text{ ft.}$$
$$\tfrac{3}{4} A = 60\text{ ft.-tons}$$
$$A = 60\text{ ft.-tons} \div \tfrac{3}{4}\text{ ft.}$$
$$A = 80\text{ tons}$$

The weight of the wall over the end of the cantilever must therefore exceed 80 tons if the latter is to be safe against overturning; or some other form of anchorage must be provided which will give an equivalent of 60 ft.-tons at F. Owing to the bonding of the

material of which the wall is composed it would be necessary to lift all the work inside the dotted lines shown in Fig. 60, and, of course, this amount would be considered in the calculations.

Beam Reactions.—Certain variations

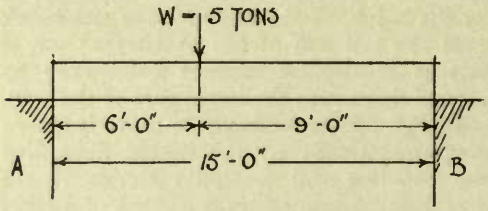


Fig. 61.—Beam Loaded Eccentrically

occur in the case of beams, as there are two reactions to consider, and these are not necessarily equal to one another. When the beam is uniformly loaded, whether the loads are concentrated or distributed, the reaction at each abutment will be equal, and their combined total must be equal to the sum of the loads on the beam. If the loading is not uniform, then the reactions will vary, and it is these cases that will be dealt with here.

Let a beam be taken with a span of 15 ft. loaded with a concentrated weight of 5 tons, situated at a point 6 ft. from one support, as shown in Fig. 61. A greater proportion of the load will be carried by the abutment A than by the abutment B, as the weight is nearer the former, and the reaction will require to be equal to the amount of the load

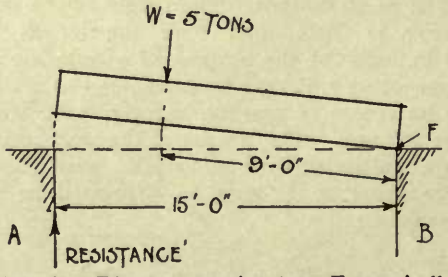


Fig. 62.—Diagram showing how Eccentrically Loaded Beam Tends to Rotate

carried. The actual amount may be calculated by again referring to the principle of the levers. In Fig. 62 it will be seen that the load has a tendency to push the beam downward and exert a pressure on abutment A by rotating on the abutment B at point F. This is an example of a lever of the third order, where the power (supplied by the load) is situated between the resistance and the fulcrum. Therefore:

$$W \times 9 \text{ ft.} = \text{resistance at A} \times 15 \text{ ft.}$$
$$5 \text{ tons} \times 9 \text{ ft.} = R^1 \times 15 \text{ ft.}$$
$$R^1 = \frac{45 \text{ ft.-tons}}{15 \text{ ft.}}$$
$$R^1 = 3 \text{ tons}$$

The reaction at A, therefore, is 3 tons, and, if the sum of the reactions is equal to the load, the reaction at B = 5 tons − 3 tons = 2 tons. This can be proved by considering the beam as rotating on abutment A to cause a pressure on abutment B, as indicated in Fig. 63. Then:

$$W \times 6 \text{ ft.} = \text{resistance at B} \times 15 \text{ ft.}$$
$$5 \text{ tons} \times 6 \text{ ft.} = R^2 \times 15 \text{ ft.}$$
$$R^2 = \frac{30 \text{ ft.-tons}}{15 \text{ ft.}}$$
$$R^2 = 2 \text{ tons,}$$

which agrees with the amount above stated.

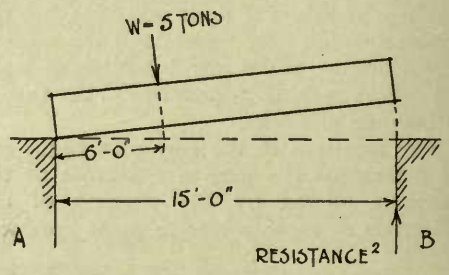


Fig. 63.—Beam Rotating on the other Abutment

A rule can now be deduced as follows: The reaction at an abutment is equal to the load multiplied by its distance from the *opposite* abutment, divided by the whole span.

If more than one load is carried, then the reaction is found by multiplying each load by its respective distance from the abutment, adding the amounts thus found, and dividing the total by the whole span; this will be made clear by an example. The example in Fig. 64 shows a beam which carries three different concentrated loads; then:

$$R^1 = [\,(3 \text{ tons} \times 15 \text{ ft.}) + (4 \text{ tons} \times 10 \text{ ft.}) + (5 \text{ tons} \times 6 \text{ ft.})\,] \div 18 \text{ ft.}$$

$$R^1 = \frac{45 + 40 + 30}{18}$$

$$R^1 = \frac{115}{18} = R^1 = 6\tfrac{7}{18} \text{ tons}$$

$$R^2 = \frac{(3\text{tons} \times 3\text{ft.}) + (4\text{tons} \times 8\text{ft.}) + (5\text{tons} \times 12\text{ft.})}{18}$$

$$R^2 = \frac{9 + 32 + 60}{18}$$

$$R^2 = \frac{101}{18} = R^2 = 5\tfrac{11}{18} \text{ tons}$$

$R^1 + R^2$ must equal total load $6\frac{7}{18}$ tons + $5\frac{11}{18}$ tons = 12 tons = total load.

Any number of loads may be dealt with in this manner, and if a distributed load over part of the length only has to be considered, it can be taken as acting at its centre of gravity for the purpose of calculating the reaction, while if it is distributed uniformly over the whole length it is obvious that one half will be carried by each support.

The weight of the beam itself has purposely been neglected with the object of simplifying the explanation, but when it is required to allow for this it should be dealt with as a distributed load as explained above.

Effective Span.—The beam diagrams in

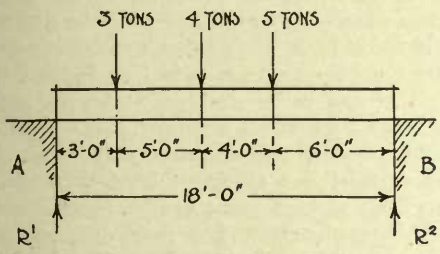


Fig. 64.—Beam Carrying Three Concentrated Loads

this and the following chapter do not show any bearing on the support. What is actually illustrated is the *effective* span, this being the distance between the centres of the bearing surface, where the pressure is theoretically considered as acting. Fig. 65 shows how the portion of the support inside the centre of pressure and the portion of the beam outside the same point are neglected, these portions being hatched. The area of the bearing surface will be proportionate to the amount of the reaction, and will vary with the material of which the support is composed; this, of course, is merely a matter of dividing the total load on the support by the safe load per unit on the material, when the required area will be given.

BENDING MOMENTS

The term "bending" needs no explanation; and the term "moment" has already been explained; it will therefore be understood that the expression "bending moment" refers to the amount or value of a force or forces to cause a tendency to bend. This value will depend on the actual amount of the force and on its disposition.

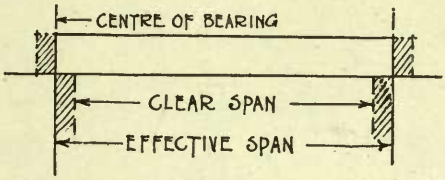


Fig. 65.—Diagram Illustrating Clear and Effective Spans

Bending Moment at Centre of Beam.—The bending moment (B M) in a beam or cantilever is caused by the load and the reaction opposing one another; and, as the beam or cantilever is the agent through which this opposition is allowed to act, it is called upon to resist a certain amount of stress due to the bending tendency.

Assume a beam to carry a load of 10 tons in the centre of its length, as in Fig. 66. The load will set into force an action which can be expressed as a *downward* action, due to the force of gravity; in order to produce equilibrium, this must be counteracted by a resistance at the abutments which is equal in amount and *opposite* in direction. Therefore, the reactions can be expressed as *upward* forces, which are set into action by the passive resistances of the abutments. It will be seen, then, that these actions (the load and the reactions) exert a bending tendency in the beam through which they act as shown (in an exaggerated manner) by

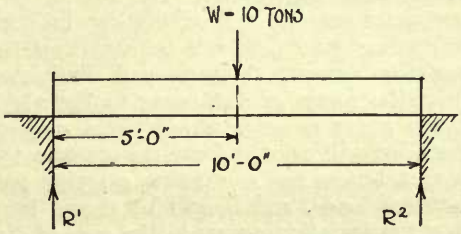


Fig. 66.—Centrally-loaded Beam

Fig. 67, in which the beam has a tendency to turn on the point F.

Now, when two parallel forces are acting in opposite directions and opposing one another, they are said to form a *couple*, and their greatest value to cause stress is the amount of *one* force multiplied by the

distance between them. In the instance just given, the greatest value is $\frac{W}{2}$ multiplied by $\frac{l}{2}$, where W equals the total weight and l equals the total span. It is obvious that

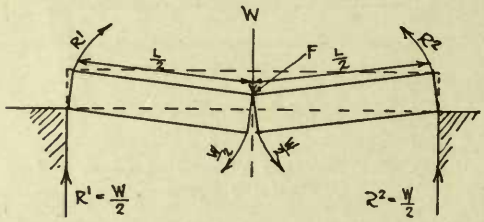


Fig. 67.—Bending Tendency on Beam

half the weight is taken by each abutment, and therefore the opposition from each end of the beam can only be due to this amount. This is explained by Fig. 67. Hence the formula for finding the greatest bending moment in the case of a beam with a *central concentrated* load is $BM = \frac{W}{2} \times \frac{l}{2}$ which equals $BM = \frac{Wl}{4}$.

This formula $\frac{Wl}{4}$ expresses the *greatest* bending moment in the case of a supported beam carrying a concentrated central load, and this greatest bending moment will be found to occur at the *centre* of the span, where moments have just been taken.

Bending Moment at any Point in Beam.—Now consider the method of ascertaining the bending tendency at any point other than the centre. By taking any particular example and calculating the B.M. at various points, it will be seen how the variation becomes uniform and diminishes from the centre of the beam to the abutment, where it becomes *nil*. This *uniform* variation will only be found to occur in the case of beams and cantilevers carrying *concentrated* loads, and it will be shown later how the variation occurs in the case of distributed loads. Assume a beam to carry a concentrated load of 10 tons at the centre of the span which is 10 ft., as before, then, according to the formula $BM = \frac{Wl}{4}$ the bending moment $= \frac{10 \times 10}{4} = 25$ ft.-tons.

As already explained, the bending tendency is due to the *reaction* multiplied by its distance to the point at which the moments are taken; and on applying this principle it will be found quite easy to ascertain the B.M. at any point. Assume the same beam with the concentrated load of 10 tons at the centre, and find the B.M. at a point 3 ft. from one support (*see* Fig. 68). Now, the reaction at each support will be equal to 5 tons, therefore, it is only necessary to multiply the reaction at A by its distance to the point at which the B.M. is required, namely, 3 ft. Hence, the B.M. = 5 tons × 3 ft. = 15 ft.-tons.

Again, the B.M. at a point 2 ft. from the support = 5 tons × 2 ft. = 10 ft.-tons, thus showing how the B.M. diminishes towards the abutment, and as the *reaction* is a *constant* figure in the calculations for the B.M. at any point, and the only factor that varies is the *distance of the point from the support*, it can be said that the B.M. at any point *varies directly as its distance from the support.*

If the B.M. at the centre is set up to scale over the beam, as shown in Fig. 69, and the span of the beam itself is drawn to scale, then the B.M. at all points will be expressed graphically by this diagram, and by scaling the vertical line at any intermediate point the B.M. at that point can be ascertained.

This explanation has now deduced the following rule:

The bending moment at any point in a beam with a concentrated load equals *the reaction at the abutment multiplied by its distance to that point.*

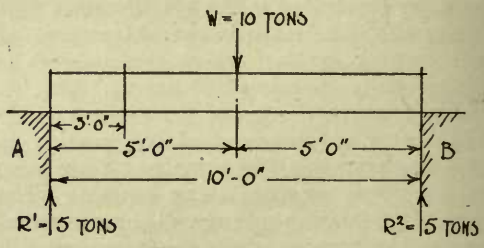


Fig. 68.—Finding Bending Moment of Beam

Weight between Reaction and Point of Calculation.—If the weight is situated *between* the reaction selected and the point round which moments are taken, then allowance must be made for the moment of the weight at this point.

This can be explained in the following manner: Again assume the same condition of loading and span and also that the B.M.

at a point 3 ft. from abutment B is required to be calculated from abutment A (see Fig. 70). It will be seen that W is situated between the abutment selected and the point at which the B.M. is to be found. Then W is situated on the arm of the lever

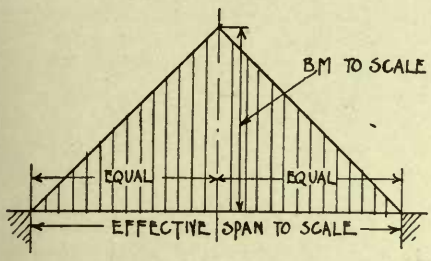


Fig. 69.—Bending Moment Set up to Scale

working from A, and the downward effect of W must be subtracted from the upward effect of R^1 at A at the point where the B.M. is to be found.

Then B M at F = R^1 × upward leverage − W × downward leverage, as shown in Fig. 71.

From this equation it will be seen that the B.M. at F actually equals (5 tons × 7 ft.) − (10 tons × 2 ft.) = 35 ft.-tons − 20 ft.-tons. = 15 ft.-tons—the same result as that obtained in the previous calculation for the B.M. at a point 3 ft. from one support with similar load and span.

The principle previously explained must be thoroughly understood, because it will be seen that in the case of a beam which

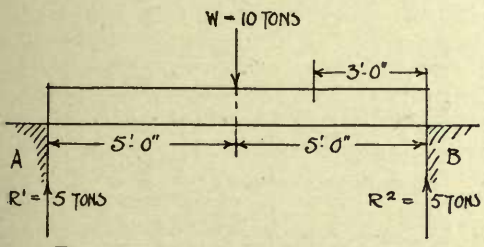


Fig. 70.—Finding Bending Moment of Beam

carries two or more concentrated loads at different points it will be quite impossible to determine the B.M. at any point between two of the loads without considering the load which is situated on the arm of the lever. Take an actual example showing these conditions.

The diagram (Fig. 72) illustrates a beam with two loads concentrated at different points, and we will assume that it is necessary to ascertain the B.M. at the centre of the beam. Then it will be necessary, whichever abutment is selected as the one to work from, to calculate with a weight between the

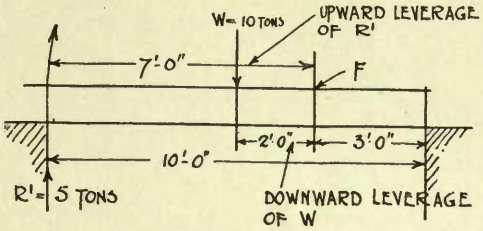


Fig. 71.—Lever Arms of Reaction and Load

reaction and the point at which moments are to be taken.

Working from abutment A, then

$$R^1 = \frac{2 \times 12 + 4 \times 6}{14}$$

$R^1 = 3\frac{3}{7}$ tons.

B M at centre = R^1 × distance to centre − W^1 × its distance to centre.

B M = $3\frac{3}{7}$ tons × 7 ft. − 2 tons × 5 ft.

B M = 24 ft.-tons − 10 ft.-tons.

B M = 14 ft.-tons.

The following rule can now be deduced: The bending moment at any point in a beam which carries one or more concentrated loads can be found by multiplying either

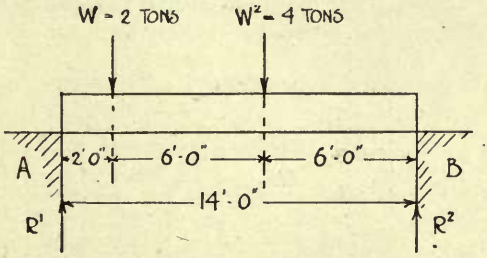


Fig. 72.—Beam with Two Loads Concentrated at Different Points

reaction by its distance to the point at which the B.M. is to be found, but if any weights are situated between the reaction selected and the point at which the moment is to be found, then a deduction must be made equal to the sum of the weights multiplied by their respective distances to this point.

This rule can be applied to any number

of loads, and once this principle is thoroughly understood no difficulty will be found with a beam carrying any number of concentrated loads, however the latter may be situated.

Bending Moment in Cantilever.—The mathematical method of determining the B.M. with cantilevers carrying concentrated loads will now be considered.

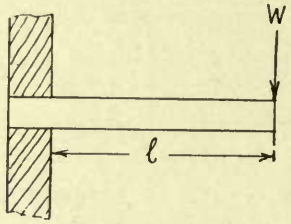


Fig. 73.—Cantilever with Concentrated Load at Outer End

Owing to the fact that only one support has to be dealt with, the calculations are much simplified, as it will be obvious that the reaction is always equal to the load; and no method is required to ascertain this as in the case of a beam.

The most simple case will be found to be that of a cantilever that carries a single concentrated load at its extreme outer end, as in Fig. 73. The greatest bending tendency due to this weight will be found to occur at the support. This bending tendency is due to the weight pressing down on the cantilever, and tending to cause failure by turning on the point F as shown in Figs. 74 and 75.

Now if the moments round this point F are taken, the moment of the weight is equal to its value in lbs., cwts., or tons × by its distance or leverage to this point. This moment, then, is equal to W × l, where W equals the weight and l the length of the cantilever.

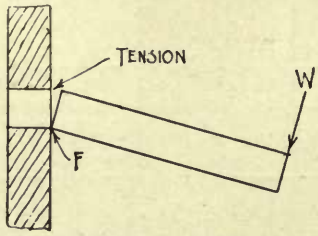


Fig. 74.—Failure of Cantilever due to Tension

Hence, the formula for finding the *greatest bending moment* in the case of a cantilever which carries an *end* load is B M = W l. The following is an example. A cantilever with a projection of 10 ft. carries an end load of 5 tons. Then B M = W l or B M = 5 × 10 = 50 ft.-tons.

To ascertain the B.M. at any intermediate point it is only necessary to apply the same principle—namely, multiply the weight by its leverage to the point at which the moment is to be found. Take the same weight and projection as in the last example, and assume that it is necessary to calculate the B.M. at a point 5 ft. from the extreme end. Then it will be seen that the bending tendency at this point is less than that at the support, as, although the weight remains the same, the *leverage* has diminished, and is now only 5 ft. instead of 10 ft., as Fig. 76. The B.M. will now equal 5 tons × 5 ft. = 25 ft.-tons.

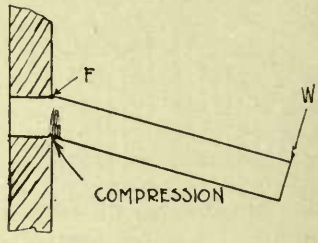


Fig. 75.—Failure of Cantilever due to Compression

A rule may now be deduced as follows: The bending moment at any point in a cantilever which carries a single load at its outer end can be found at any point by multiplying the load by its distance to that point.

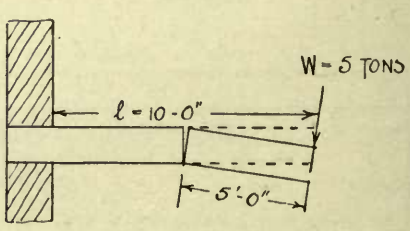


Fig. 76.—Bending Moment in Cantilever

It will be seen that one factor in the calulations always remains the same—namely, the weight, and it is only the distance or leverage that varies. It has been shown that the greatest B.M. occurs at the support, and it will be obvious that there is no bend-

ing tendency at the extreme outer end, as the weight at this point will have no leverage. Therefore, the greatest B.M. will occur at the support, and it will diminish from this point to the outer end, where it is nil. It is also known that the diminution will be uniform, as it is dependent *only* on the distance from the support. Therefore, if the B.M. at the support is set up to scale at this point and a straight line is drawn through to the outer end, a diagram will be obtained which will represent the B.M. to scale at all points in the cantilever due to the end load, as Fig. 77.

Bending Moment in Cantilever carrying more than one Concentrated Load. —In the case of a cantilever which carries two or more concentrated loads at different

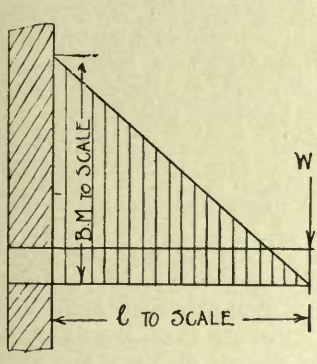


Fig. 77.—Bending Moment of Cantilever Set out to Scale

points, it is only necessary to apply the same principle of multiplying the loads by their respective leverages at any particular point, and the sum of their values will give the bending moment. There are one or two points, however, to be borne in mind, and the chief of these is the fact that a load does *not* cause a bending tendency at any point which is *not* situated between this *load* and *the support.*

An illustration will render this quite clear. In Fig. 78 the load represented by W will only cause a bending tendency in that part of the cantilever marked A, and *no* bending tendency whatever in the part marked B, which is *not* between the load and the support, and this is obvious, as the load will travel *inwards* to the support over the portion A to meet the reaction, and there will be no tendency to travel outwards over B, as no resistance is offered by the outer end.

Take an example with two concentrated loads and calculate the B.M. at different points. This will show how the moments are to be found.

The diagram, Fig. 79, shows two loads of 2 tons and 4 tons respectively. The B.M. at the support will be due to each weight

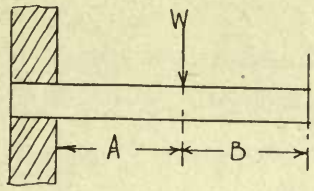


Fig. 78.—How Centrally-loaded Cantilever Tends to Bend

multiplied by its respective leverage, and these must be added, as they are *not opposing* forces, but forces acting together.

Then B M at support $= (W^1 \times 6 \text{ ft.}) + (W^2 \times 9 \text{ ft.}) = 2 \text{ tons} \times 6 \text{ ft.} + 4 \text{ tons} \times 9 \text{ ft.} = 12 + 36.$

B M $= 48$ ft.-tons.

As a weight will not cause any bending tendency at a point which is not between the weight and the support, W^1 will not cause any B.M. between W^1 and W^2, and the only tendency will be due to W^2, as Fig. 80.

B M under W^1 then equals $W^2 \times 3$ ft. $=$ 4 tons $\times$ 3 ft. $=$ 12 ft.-tons.

Bending Moment in Beam carrying Distributed Load.—Concentrated loads only have been dealt with up to the present, and although the same principles apply in the case of distributed loads a few brief

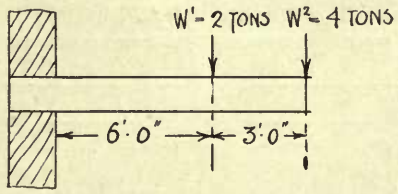


Fig. 79.—Cantilever with Two Concentrated Loads

notes should be of value, more especially as distributed loads are more generally met with in practice. A common case that has to be calculated is that of a beam carrying both concentrated and distributed loads, and an example of this nature presents great difficulty to one who is not well versed in structural design.

As a first example, consider a beam which carries a uniformly distributed load over its whole length, as shown in Fig. 81. Then the total load will be the length multiplied by the weight per foot run. This equals 10 ft. × 1 ton per foot = 10 tons. The reactions

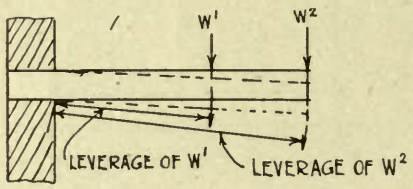


Fig. 80.—Bending Tendency in Cantilever with Two Concentrated Loads

will obviously be equal, and must together equal the total weight, which may be expressed as W. Then R^1 and R^2 each = $\frac{W}{2}$. The bending moment at any point is equal to the reaction multiplied by its distance to that point, minus any weight situated between the reaction and the point at which the bending moment is to be found multiplied by its distance to the same point. The greatest bending moment will be at the centre of the span, and upon reference to Fig. 82 it can be seen what values have to be taken.

There R^1 equals $\frac{W}{2}$, and this acts upwards with a leverage of $\frac{l}{2}$ at the point F, which is the centre of the span. There is, however,

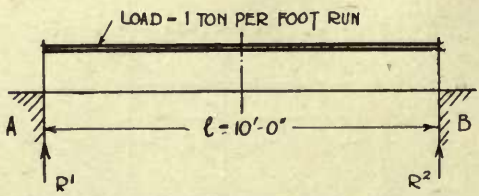


Fig. 81.—Beam with Uniform Distributed Load

a load on the lever arm which is acting downward, and this load is equal to $\frac{W}{2}$, and being uniformly distributed, it can be considered as acting at its centre of gravity, which will be situated at a distance equal to $\frac{l}{4}$ from point F.

The greatest bending moment then equals

$$\frac{W}{2} \times \frac{l}{2} - \frac{W}{2} \times \frac{l}{4}$$
$$= \frac{Wl}{4} - \frac{Wl}{8} = \frac{Wl}{8}$$

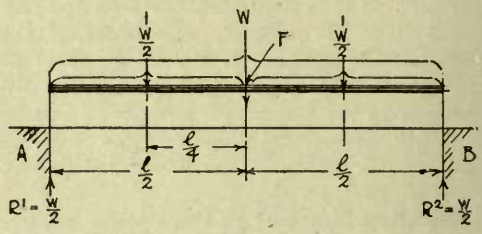


Fig. 82.—Distributed Load Acting through Centre of Gravity

This is the formula for the *greatest* bending moment in the case of a beam which carries a *uniformly distributed load* over its whole length. To work out the example in Fig. 81 it is only necessary to apply the formula as follows:

$$\text{G B M} = \frac{Wl}{8} = \frac{10 \times 10}{8} = \frac{100}{8} = 12{\cdot}5$$

ft.-tons.

Let it be now assumed that the bending moment is required to be calculated at a point 3 ft. from abutment A.

Then the length of the lever arm is reduced to 3 ft., and in addition the load acting downward is reduced to 3 tons with a leverage to the point F of 1 ft. 6 in., as in Fig. 83. The bending moment equals:

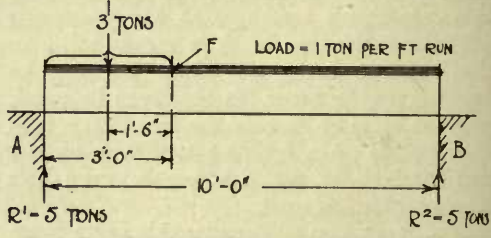


Fig. 83.—Bending Moment at Intermediate Point

(5 tons × 3 ft.) − (3 tons × 1·5 ft.)
= 15 ft.-tons − 4·5 ft.-tons
= 10·5 ft.-tons.

Although the reaction, which is one of the factors in the calculations for any intermediate point, remains the same, the leverage diminishes towards the support, and in addition to this, the weight acting down-

ward, together with its leverage, diminishes as the lever arm of the reaction becomes less. The diminution of the bending moment from the centre of the span to the support is therefore not regular, as in the case of a central concentrated load, but actually varies in a compound ratio. If the bending moment at the centre be set up to scale over the beam, then the parabolic curve, as shown in Fig. 84, will give the bending moment at all intermediate points. With regard to a combination of distributed and concentrated loading, an example is given in Fig. 85, and, if this is explained, no difficulty should be experienced with other examples, as the principles are

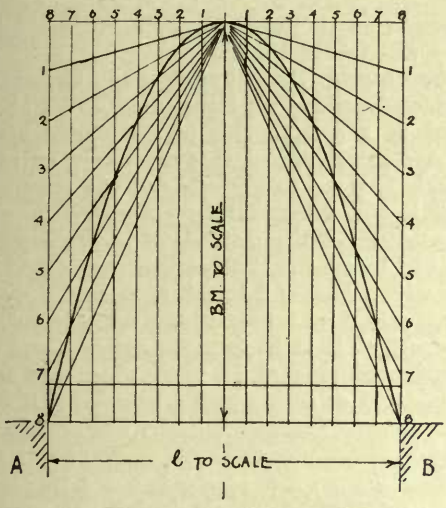


Fig. 84.—Bending Moment on Beam Set up to Scale

always the same, and the procedure is quite simple.

First calculate the reactions:

$$R^1 = \frac{6 \text{ tons (distributed load)} \times 9 \text{ ft.} + 8 \text{ tons} \times 4 \text{ ft.}}{12 \text{ ft.}}$$

$$R^1 = \frac{54 + 32}{12}$$

$$R^1 = \frac{86}{12} = 7\tfrac{1}{6} \text{ tons}$$

$$R^2 = \frac{8 \text{ tons} \times 8 \text{ ft.} + 6 \text{ tons} \times 3 \text{ ft.}}{12 \text{ ft.}}$$

$$R^2 = \frac{64 + 18}{12}$$

$$R^2 = \frac{82}{12} = 6\tfrac{5}{6} \text{ tons.}$$

Assume that the bending moment at F under the load of 8 tons is to be found by working from abutment A, then:

B M $= (7\frac{1}{6}$ tons $\times$ 8 ft.) $-$ (6 tons $\times$ 5 ft.)

B M $= 57\frac{1}{3}$ ft.-tons $- 30$ ft.-tons $= 27\frac{1}{3}$ ft.-tons. This can be checked by working from abutment B. Then:

B M $= 6\frac{5}{6}$ tons $\times$ 4 ft. $= 27\frac{1}{3}$ ft.-tons.

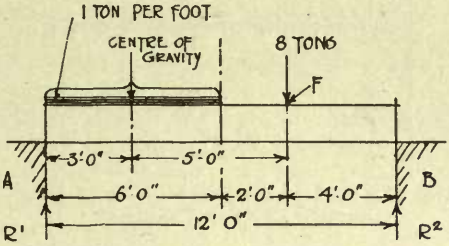


Fig. 85.—Beam with Concentrated and Distributed Loading

The bending moment at any other point can be found in a similar manner, and, if necessary, a diagram set up showing the value at all parts of the beam.

Bending Moment in Cantilever Carrying Distributed Load.—In the case of a cantilever carrying a uniformly distributed load over its whole length, the greatest bending moment occurs at the support, and the method of finding its value is very simple.

The portion of the weight at the *extreme outer* end of the cantilever will have a leverage equal to l, while the portion of the weight at the *extreme inner* end will have a leverage equal to nil. The mean leverage may there-

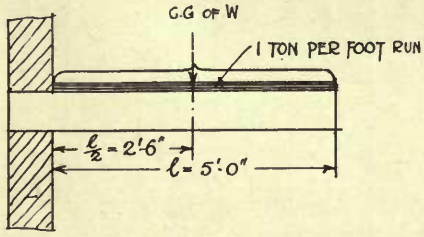


Fig. 86.—Cantilever Carrying Distributed Load

fore be assumed to be $\frac{l}{2}$, which represents distance from the support to the centre of gravity of the weight. The formula for the greatest bending moment then becomes:

$$W \times \frac{l}{2} = \frac{Wl}{2}$$

In the example in Fig. 86, the greatest

bending moment is equal to the weight of 5 tons multiplied by $\frac{l}{2}$, which is 2 ft. 6 in.; this then gives a moment of 5 × 2 ft. 6 in. = 12½ ft.-tons.

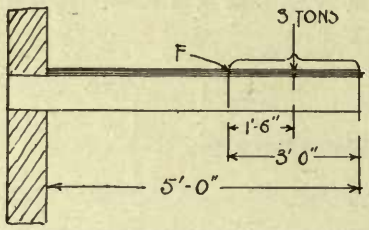


Fig. 87.—Finding Bending Moment at Intermediate Point in Cantilever

In the case of the bending moment at any intermediate point, it is only necessary to consider that portion of the weight situated between the point and the extreme outer end. As an example, assume that the bending moment is to be calculated at the point F in the cantilever shown in Fig. 87. Then the weight to be considered is 3 tons, and

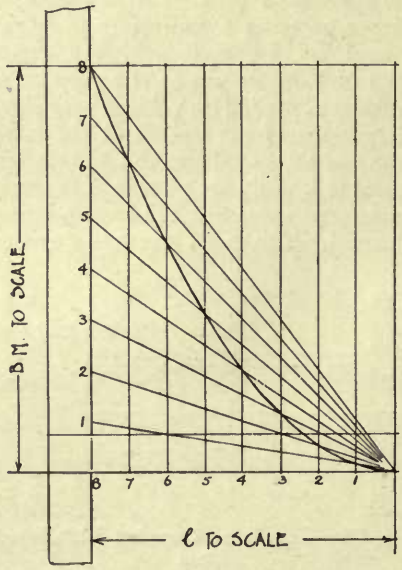


Fig. 88.—Bending Moment in Cantilever Set out to Scale

acting at its centre of gravity it will have a leverage of 1 ft. 6 in. to the point F. The bending moment = 3 tons × 1 ft. 6 in. = 4½ ft.-tons. The moment varies in a compound ratio, as in the case of a beam, owing to the weight diminishing, because the point at which the moment is found is moved outward from the support, and at the same time the lever arm also decreases.

If the bending moment at the support is set up to scale as illustrated in Fig. 88, a diagram can be drawn to show the bending

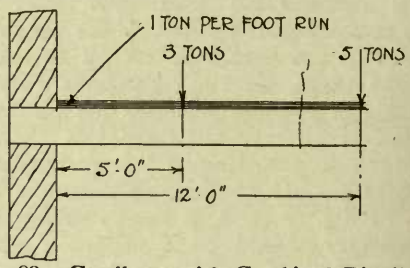


Fig. 89.—Cantilever with Combined Distributed and Concentrated Loading

moment at all points in the cantilever due to a distributed load.

Combined Distributed and Concentrated Loading on Cantilever.—With regard to a combination of distributed and concentrated loading on a cantilever, an example is given in Fig. 89, and the greatest bending moment will be calculated as an illustration of the method to be employed. It will be seen that three loads are carried, namely, a load of 3 tons 5 ft. from the support, a load of 5 tons at the extreme outer end, which is 12 ft. from the support, and a distributed load of 12 tons, which can be considered as acting with a leverage of 6 ft. The greatest bending moment will therefore be :—

(3 tons × 5 ft.) + (5 tons × 12 ft.) + (12 tons × 6 ft.)
= 15 ft.-tons + 60 ft.-tons + 72 ft.-tons
= 147 ft.-tons.

The bending moment at any other point can be found by considering only that portion of the loading which is situated between the point and the outer end of the cantilever.

MOMENT OF INERTIA

The moment of inertia is extremely important in all cases of advanced calculations. The expression is used to define the relative values of sections of different shapes. From the moment of inertia, the section modulus and moment of resistance (two terms that will be explained later) can be found.

Explanation of Moment of Inertia.—The moment of inertia of a section is an indication of the strength of that section. It is a constant value, and depends on the shape of the section. The area alone

is no indication of the strength of a section from a structural point of view. Two sections of the same area do not necessarily have the same power to resist stress. For example, place a piece of timber, 9 in. by 2 in. in section, as a beam over a certain span—say 10 ft.—it will have a certain amount of power to resist stress, or, in other words, a certain amount of power to remain "inert," which power can be expressed as its "moment of inertia." The timber will have two values, according to the way in which it is placed. In the form of a beam 9 in. deep and 2 in. wide, it will be much stronger as a weight-carrying member than as a beam 2 in. deep and 9 in. wide. This can be proved by a practical demonstration quite easily, or by the theory of the moment of resistance for a rectangular beam, to be given later.

The reason for this difference in strength with the same section is entirely due to the disposition of the fibres in relation to the neutral axis of the section, which is different in each case. (See page 3 for an explanation of what is meant by the term "neutral axis.") In the first instance, the beam was 9 in. deep, and therefore some of the fibres were situated at a distance of 4½ in. from the neutral axis, whereas in the second instance the beam was only 2 in. deep, and therefore the extreme fibres were only 1 in. from the neutral axis. Now the stress in a beam is nil at the neutral axis, and it increases in intensity as it gets farther away from the neutral axis. This fact allows the fibres that are at a distance from the neutral axis to be utilised to their fullest capacity, whereas those adjoining the neutral axis cannot be called upon to exert their greatest resistance. Again, it will be shown later that the lever arm of the tensional and compressional areas is increased as the depth is increased, and this greatly increases the value of the resistance offered.

These remarks should suffice for the present to show that the area of a section alone is no criterion as to the strength of the member, and that the same section will possess two different capacities according to the way in which it is placed.

It will now be understood why the two values for the moment of inertia of a section are known as the "least moment of inertia" and the "greatest moment of inertia," according to the neutral axis around which the "moment" has been taken. In the case of a section which is square or circular there will only be one value for the moment of inertia, as the axis will be at the same distance from the extreme fibres in both cases, except in the case of the square section being placed on one edge, which is so unusual in structural work that it need not be considered for the present. The moment of inertia in value will be dependent, in every case, on the *area* of the section, and the *disposition* of that area in relation to the neutral axis of the section.

Calculating Moment of Inertia.—The method of finding the moment of inertia of a plane figure is based on the following principle. The section or plane surface is

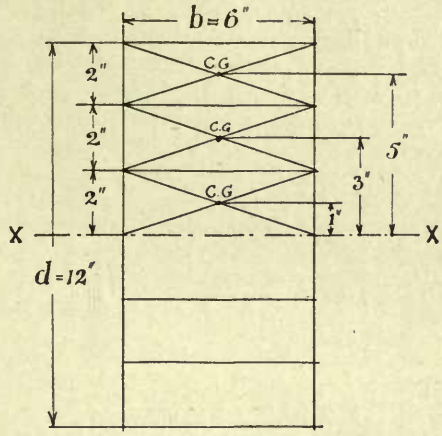


Fig. 90.—Calculating Moment of Inertia of Simple Rectangular Section

imagined to be divided into an infinite number of thin layers, and the area of each layer is multiplied by the square of the distance between the centre of gravity of the layer and the axis; the sum of all these products is the moment of inertia of the plane surface with respect to that axis.

An attempt should now be made to calculate the moment of inertia of a simple rectangular section as given in Fig. 90. Assume that the section is 12 in. deep and 6 in. wide, and the moment of inertia is to be taken about the axis x—x. Also assume that there are three layers on each side of the axis 2 in. thick. Then the moment of inertia will equal

$$[(6 \times 2 \times 5^2) + (6 \times 2 \times 3^2) + (6 \times 2 \times 1^2)] \times 2 = [300 + 108 + 12] \times 2 = 420 \times 2 = 840.$$

The presence of fifty layers on each side of

the axis, instead of three, could be assumed, and thus a great amount of time could be tediously spent in calculating the "moment." A formula has, however, been devised for the moment of inertia of all the simple sections, and the formula will be given for each case.

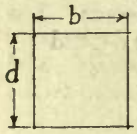


Fig. 91

$$I = \frac{b\,d^3}{12}$$

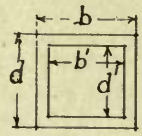


Fig. 92

$$I = \frac{b\,d^3 - b'\,d'^3}{12}$$

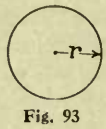


Fig. 93

$$I = \cdot 7854\, r^4$$

Fig. 94

$$I = \cdot 7854\,(r^4 - r'^4)$$

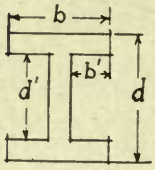


Fig. 96

$$I = \frac{b\,d^3 - 2b'\,d'^3}{12}$$

Figs. 91–97 Common Sections and their Moments of Inertia

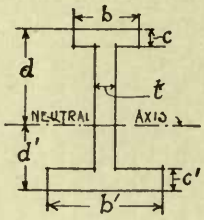


Fig. 97

$$I = \tfrac{1}{3}\left\{b\,d^3 - (b - t)\,(d - c)^3 + b'\,d'^3 - (b' - t)\,(d' - c')^3\right\}$$

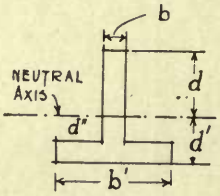


Fig. 95

$$I = \tfrac{1}{3}\left\{b\,d^3 + b'\,d'^3 - (b' - b)\,d''^3\right\}$$

Now the formula for a rectangle is $\frac{b\,d^3}{12}$, and for a square $\frac{b^4}{12}$, and so the true value of the inertia moment for the section given in Fig. 90 will be $I = \frac{b\,d^3}{12}$.

$$I = \frac{6 \times 12^3}{12} = 864.$$

It will be seen that there is some difference in the result obtained by the formula and that obtained previously, this being due to the fact that the number of layers in the first instance was taken at three only. The moments of inertia of the common sections are as given by Figs. 91 to 97.

In each case these formulæ give the least moment of inertia in the positions shown, and as an example a rolled steel joist will be taken and the moment of inertia calculated (*see* Fig. 98).

The moment of inertia will be more about the axis X—X than about Y—Y (Fig. 99). The formula will be $I = \frac{b\,d^3 - 2\,b'\,d'^3}{12}$

$$= \frac{6 \times 12^3 - 2 \times 2{\cdot}75 \times 10{\cdot}5^3}{12} = 333{\cdot}421.$$

To calculate the moment about the other axis, place the joist so that it is 6 in. deep and 12 in. wide, as Fig. 99. The inertia will be

$$\frac{\overbrace{\cdot 75 \times 6^3 \times 2}^{\text{flanges}} + \overbrace{10{\cdot}5 \times \cdot 5^3}^{\text{web}}}{12} = 27{\cdot}109.$$

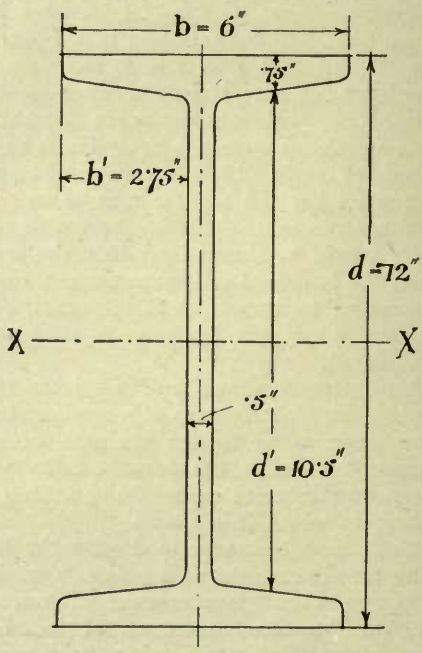


Fig. 98.—Calculating Moment of Inertia of Rolled Steel Joist

SECTION MODULUS AND MOMENT OF RESISTANCE

The terms "section modulus" and "moment of resistance" are often used indiscriminately, but although they are closely allied, they are distinct terms, and have different values.

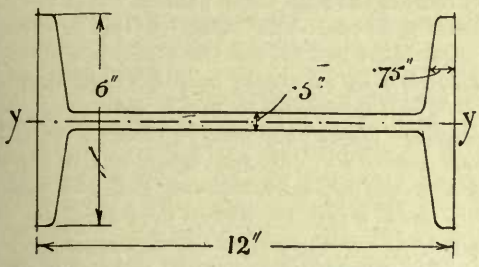


Fig. 99.—Calculating Least Moment of Inertia of Rolled Steel Joist

The *section modulus* can be found by dividing the moment of inertia by half the depth of the section.

The *moment of resistance* can be found by multiplying the section modulus by the modulus of rupture of the particular material under consideration.

Thus, the section modulus is a fixed quantity for any given section, whilst the moment of resistance depends on the nature of the material employed.

The bending moment or tendency to cause failure in a member must be equalled or exceeded by the moment of resistance of that member; and to illustrate this fact, and also how the section modulus is obtained —and from this the moment of resistance— will be the next step.

Calculating Section Modulus.—It has already been shown that the reaction and the weight on a beam causes a certain rotating tendency, as indicated in Fig. 100. It has also been shown that the value of these external forces on the beam is equal to the amount of one force multiplied by its distance from the other force, as they are equal, opposite, and parallel forces forming a "couple."

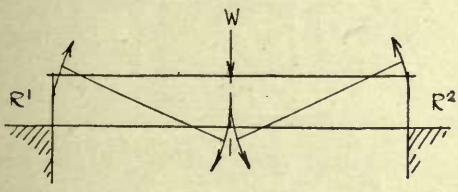


Fig. 100.—Rotating Tendency of Beam, caused by Reaction and Weight

Take one-half of the beam and imagine the forces to be acting upon it, as shown in Fig. 101. Then the rotating tendency caused by the external forces must be equalled by the internal forces having a rotating tendency in the opposite direction. This rotating tendency of the internal forces is offered by the compressional and tensional resistances of the beam, which are equal to one another, opposite in direction, and parallel, as shown in the diagram by C and T. Then these two resistances form a "couple," and their value to resist the bending moment will equal the amount of one resistance multiplied by their distance apart. The method of ascertaining this value will be shown.

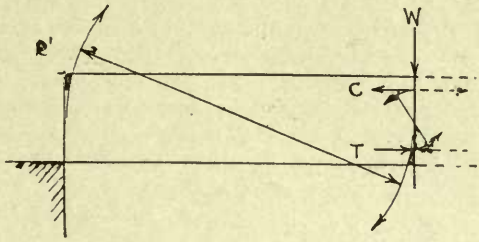


Fig. 101.—Forces Acting upon One-half of Beam

It will be seen that the rotating tendency of the reaction and weight is in a "clockwise" direction, whilst the tendency of the internal forces is "anti-clockwise." The direction of the tension and compression arrows, shown on the diagram, can be proved in the following manner. The firm arrows indicate the fibres on the half of the beam under consideration, and the dotted arrows show those on the other half of the beam. The top fibres are being compressed or pushed together, and therefore the tendency of the resistance is to pull the fibres apart as shown, and vice versa in the lower part of the beam.

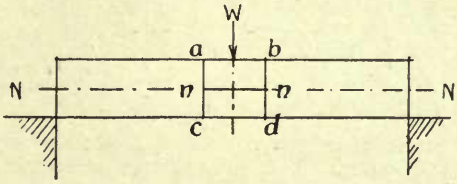


Fig. 102.—Calculating Section Modulus: Beam Centrally Loaded

The amount of the rotating tendency of the internal forces can be reasoned as follows. Assume that a beam carries a central load, as Fig. 102, and mark a piece of the beam *a b c d* as shown. The tendency

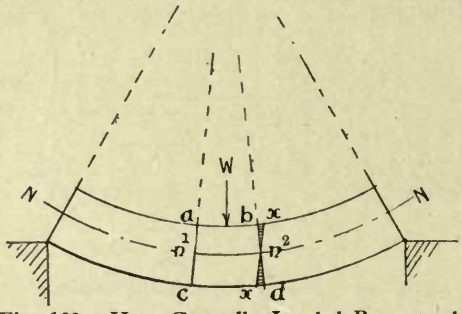


Fig. 103.—How Centrally Loaded Beam tends to bend

of the load is to cause the beam to bend in the middle, and this is shown in an exaggerated manner in Fig. 103. Now mark on the beam the same portion *a b c d* as shown on the previous diagram. In order to do this, begin by marking the length *n n* on the neutral axis, as it is known that no change has taken place in the length of the fibres at this point, there being no compression or tension. Through these points *n n* draw the lines *a c* and *b d*, which must, of course, radiate to the same centre as that from which the curved beam was struck. It will then be seen that *a c* and *b d* are no longer parallel to one another as in the original beam, or, in other words, the distance between *a* and *b* has become shorter owing to compression and the distance between *c* and *d* has become longer owing to tension.

Now, if a line *x x* be drawn through n^2 parallel to the line *a c*, as shown on the diagram in Fig. 103, the distance *b x* will equal the amount of compression that has taken place in the extreme fibres at the top and the distance *x d* will indicate the amount of tension that has taken place in the extreme bottom fibres. The stress in the extreme fibres must not exceed the resistance of the material, as otherwise failure would result. This will demonstrate the fact that as the extreme fibres only can be stressed to the safe resistance, the full value of the other fibres can never be realised, and the value of each layer of fibres decreases as it approaches the neutral axis.

This principle can now be applied to the section of a beam and the value of the compression or tensional areas ascertained. Take the section of a beam 12 in. by 6 in., as shown in Fig. 104, where the intensity of the stress is indicated by the graduation of the lines, which become fainter as the stress becomes less. Assume also that the limiting stress on the extreme fibres is 5 tons per square inch. Then the stress at the neutral axis is nil, and the mean stress on the fibres is $2\frac{1}{2}$ tons per square inch. The resistance must be equal to the stress if there is to be equilibrium, and therefore the resistance of the compressional area will be 6 in. × 6 in. × $2\frac{1}{2}$ tons per square inch = 36 × $2\frac{1}{2}$ = 90 tons.

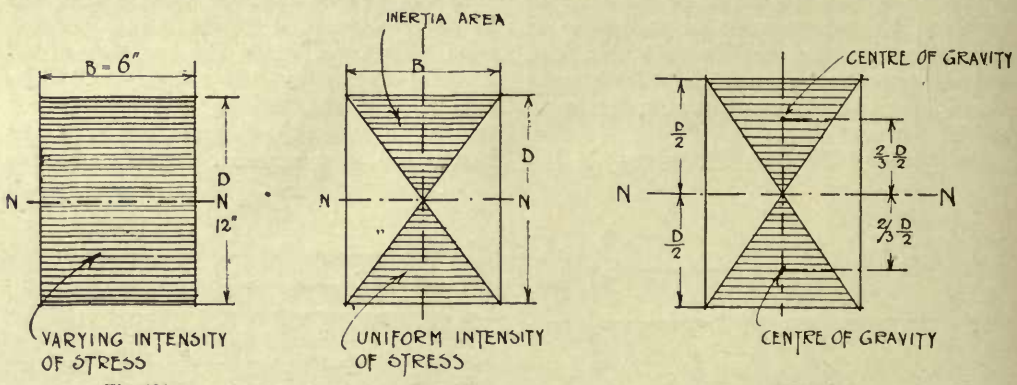


Fig. 104 **Fig. 105** **Fig. 106**

Figs. 104 to 106.—Calculating Section Modulus of Beam

Next, instead of assuming that the stress on the fibres varies in *intensity* from the outer edge to the neutral axis, assume that the intensity is the same, but the area over which it acts varies from the outer edge to the neutral axis as shown by the shaded triangles in Fig. 105, and it is found that the

same result is given, with the same sectional area of beam. Here the shaded triangle which shows the compressional area is equal to one-quarter the sectional area of the beam, or, in other words, $\frac{b\,d}{4}$. Afterwards multiply this by the uniform maximum intensity of 5 tons per square inch, then

$$\frac{6 \times 12}{4} \times 5 \text{ tons per square inch}$$

$$= \frac{72}{4} \times 5 = \frac{360}{4} = 90 \text{ tons}$$

as before. This area $\frac{b\,d}{4}$ is called the equivalent inertia area, and it simplifies the calculations to consider the beam as offering its resistance in this way.

There are two resistance areas, namely, the compressional and the tensional, and it will be seen that they are equal. They represent the internal forces which are opposite in direction, and their value to resist stress is the amount of one multiplied by its distance from the other, as they form a "couple." The distance apart of the two forces will be the distance between their centres of gravity as they are represented by triangles, and the centre of gravity of a triangle is situated at a distance of two-thirds of its height from the apex, as shown in Fig. 106. The centres of gravity are then each $\frac{2}{3}\,\frac{d}{2}$ from the neutral axis, and their distance from one another is $\frac{2}{3}\,\frac{d}{2} + \frac{2}{3}\,\frac{d}{2}$ which equals $\frac{2}{3}\,d$.

The distance $\frac{2}{3}\,d$ then equals the lever-arm of the internal forces. The area of one force $= \frac{b\,d}{4}$ and the area multiplied by the lever-arm will give the value of the section. This can be expressed as

$$\frac{b\,d}{4} \times \frac{2}{3}\,d = \frac{2\,b\,d^2}{12} = \frac{b\,d^2}{6}$$

The value $\frac{b\,d^2}{6}$ then gives the value of a rectangular section, or, in other words, its "section modulus." It was stated previously that the moment of inertia divided by half the depth of the beam = the section modulus. The moment of inertia for a rectangular beam is $\frac{b\,d^3}{12}$. Then $\frac{b\,d^3}{12} \div \frac{d}{2}$ $= \frac{b\,d^2}{6}$ as before.

The section modulus is usually expressed in formulæ by the letter Z.

Calculating Moment of Resistance.—Having ascertained the "section modulus" and followed the principle of the action of the internal forces, the actual resistance of these forces, which not only depends on the area and leverage, but also upon the strength of the material, has to be considered.

Up to the present, no account has been taken of the strength of the material, and therefore the section modulus for any shape section will be constant, no matter what material is employed, and this is a great consideration.

Thus $\frac{b\,d^2}{6}$ is the section modulus for a rectangular section, and the moment of resistance can be found from this, according to the strength of the material employed, by multiplying the section modulus by the ultimate or safe strength of the material per square inch, according to whether the actual or safe resistance is wanted.

As an example, ascertain the safe resistance of a rectangular steel beam 8 in. × 2 in.

Then $\text{M R} = \frac{b\,d^2}{6} \times 7\frac{1}{2}$ tons, or $\text{M R} = \frac{f\,b\,d^2}{6}$, where f equals the limiting stress on the fibres, which is $7\frac{1}{2}$ tons per square inch.

Then $\text{M R} = \frac{2 \text{ in.} \times 8 \text{ in.} \times 8 \text{ in.} \times 7\frac{1}{2} \text{ tons}}{6}$

M R = 160 in.-tons. This, of course, must equate with the bending moment to produce equilibrium.

It has been seen that the moment of inertia divided by half the depth of the section gives the section modulus, and the section modulus multiplied by the limiting stress on the fibres gives the moment of resistance. This is expressed in the following formula, which is one greatly used in practice: $\text{M R} = f\frac{\text{I}}{y}$, where f = limiting stress on the fibres, I = moment of inertia, and y = half depth of section. Upon working out the example just given by this method, the same result will be obtained.

CALCULATING SAFE LOAD ON BEAM

The following example should assist the student to understand the application of formulæ already given. Find the central load that can be safely carried over a span of 16 ft. by a rolled-steel joist 12 in.

× 6 in., with flanges and web 1 in. thick.

$$MR = f\frac{I}{y}, \quad f = 7\tfrac{1}{2} \text{ tons}, \quad I = \frac{b\,d^3}{12} - \frac{2\,b'\,d'^3}{12}$$

$$= \frac{6 \times 12 \times 12 \times 12}{12} - \frac{2 \times 2\tfrac{1}{2} \times 10 \times 10 \times 10}{12}$$

$$= 864 - 416{\cdot}6 = 447{\cdot}4$$

$$y = \frac{d}{2} = \frac{12}{2} = 6 \text{ in.}$$

$$\text{then } MR = \frac{7{\cdot}5 \times 447{\cdot}4}{6}$$

$$MR = 559{\cdot}25 \text{ in.-tons.}$$

But $BM = MR$, and $BM = \frac{w\,l}{4}$ for central load.

$$\text{Therefore } \frac{w\,l}{4} = 559{\cdot}25 \text{ in.- tons}$$

$$l = 16 \times 12 = 192 \text{ in.}$$

$$\frac{w \times 192}{4} = 559{\cdot}25$$

$$w = \frac{559{\cdot}25 \times 4}{192}$$

$$w = 11{\cdot}65 \text{ tons.}$$

This method is absolutely accurate, and is not approximate, as would be the case with an empirical formula. The student is strongly advised to work out several examples himself, which will familiarise him with the use of the formula, and impress it upon his memory. A good method is to select several rolled-steel joists from a manufacturer's list, calculate the safe loads for various spans, and then check the results by reference to the table of safe loads for the various joists invariably given in the list of sections.

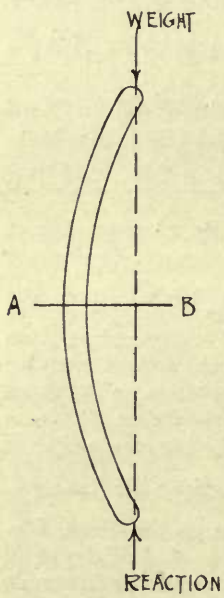


Fig. 107.—Strut under Load

COLUMNS AND STRUTS

The theory of long columns and struts unfortunately is in an unsatisfactory condition, as there is a great difference of opinion as to the best formula to use; and as the formulæ in general practice give very varying results, the student is often confused and doubtful as to the best course to adopt. Most of the formulæ in use are of a more or less complicated nature, and practically all are empirical formulæ, containing some factor which has been derived from experiment, and which must be taken

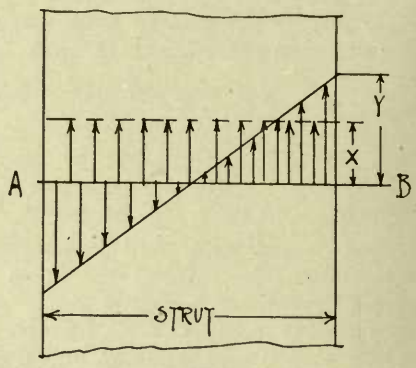


Fig. 108.—Diagram Indicating Stress in Part of Strut

for granted. Unlike a tension bar, a compression bar which is under a certain stress is apt to bend, and this bending tendency has to be resisted, in addition to the compression proper, as shown in Fig. 107.

Theoretically, with the stress passing absolutely through the centre line of the strut there is no tendency to bend, but practically this condition is never fulfilled, and the slightest deviation from the exact centre will cause a bending stress. The diagram in Fig. 108 shows a piece of the strut at A B enlarged, and the arrows indicate the stress. The horizontal line represents the compression proper, with an amount equal to X, while the sloping line indicates the stress due to the bending tendency, and this, of course, will vary in intensity at different points in the section; the edge of the strut at B will be under great compression, while the fibres at A will be in tension. Now the maximum intensity of compression will be at B, where the stress will equal $x \times y$, while at A the stress will be at a minimum, and will be equal to $x - y$, and if y is greater than x, then the fibres at this point will be in tension. It is clear that the greatest stress $x + y$ must not be greater than the safe resistance to compression of the material. The principal difficulty occurs in the determination of the bending tendency.

Struts and columns are usually divided into "long" and "short" compression bars, and in the short bars the calculations are based on the assumption that the failure is

by direct crushing only, while long bars are those which fail by bending. A sudden change has been assumed, although, of course, this does not actually happen. The following is the ratio between the least diameter and the length when the bars cease to be considered as short. Wood, 20; wrought-iron and steel, 10; cast-iron, 5. These ratios will be found to vary greatly according to different authorities.

The formula chiefly used in this country for long struts is that known as "Gordon's Formula," which is as follows:—

$$R_c = \frac{A\,r_c}{1 + a\dfrac{l^2}{d^2}}, \text{ where}$$

a = a constant found by experiment.
d = the least diameter.
A = area of the cross section.
r_c = safe resistance to compression of the material.
l = length in same units as d.
R_c = total pressure in same units as r_c.

Material	*Cross Section*	*Values of a*			r_c
		Ends hinged	*Ends fixed*	*1 hinged 1 fixed*	
TIMBER ..	Rectangular Circular ..	$\frac{4}{250}$	$\frac{1}{250}$	$\frac{1}{100}$	13 cwt.
WROUGHT-IRON	Rectangular Circular .. Solid .. Hollow ..	$\frac{4}{2500}$	$\frac{1}{2500}$	$\frac{1}{1000}$	4 tons
	L T + □ I I ⊔	$\frac{4}{900}$	$\frac{1}{900}$	$\frac{1}{360}$	
STEEL ..	do.	do.	do.	do.	7½ tons
CAST-IRON	Circular Solid ..	$\frac{1}{100}$	$\frac{1}{400}$	$\frac{1}{160}$	8 tons
	Circular Hollow ..	$\frac{1}{200}$	$\frac{1}{800}$	$\frac{1}{320}$	
	Rectangular	$\frac{3}{400}$	$\frac{3}{1600}$	$\frac{3}{640}$	
	X shaped ..	$\frac{3}{200}$	$\frac{3}{800}$	$\frac{3}{320}$	

In r_c a factor of safety of 4 only is allowed, and therefore, in the case of timber and cast-iron, it will be advisable to use a higher factor in important work, and this will reduce the value of r_c.

SHEARING STRESS

Before studying calculations for the value of the shearing stress under different conditions of loading, it will be advisable to explain what the expression "shearing

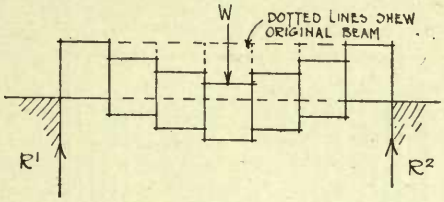


Fig. 109.—Beam, under Vertical Shear, Assumed to Consist of Separate Blocks

stress" means. This stress is actually a sliding tendency caused by the opposition of the load and the reaction, and as the beam or cantilever is the medium through which these two can oppose one another, the beam is subjected to this tendency both vertically and horizontally. As an illustration of this, let it be assumed that it is possible to divide a beam up into a certain number of parts by cutting vertically through it, so that each block is separate from the adjoining block, and yet it is possible to put the beam in position and apply the load. Then the load, in travelling along the beam toward the abutments would tend to push down each block and cause it to slide past the adjoining block as illustrated in Fig. 109. This does not actually take place in practice, as the beam would be homogeneous; but the tendency is present, and must be provided for. This is known as vertical shear. There is also

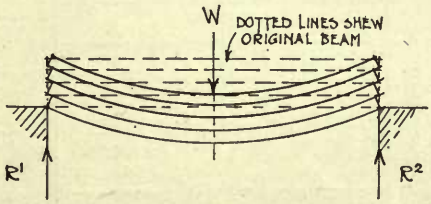


Fig. 110.—Beam, under Horizontal Shear, Assumed to consist of Separate Planks

a tendency to shear in a horizontal manner, and this is illustrated in Fig. 110, where the beam is divided into a number of horizontal parts or planks; upon the load being applied, there is a tendency for the beam to assume the form shown in an exaggerated

manner. As the ends of the planks coincided in the first instance, and after the load has been applied they are no longer coincident, it is obvious that they have slid past one another, thus forming a hori-

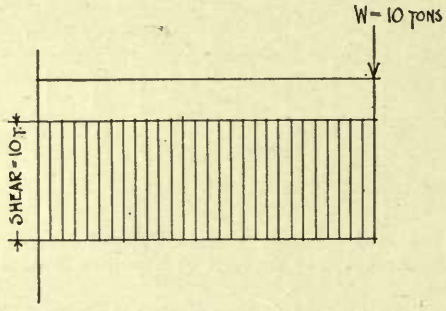


Fig. 111.—Shearing Stress in Cantilever which carries Concentrated Load at Outer End

zontal shearing action. At all points in the beam the vertical and horizontal shearing stress are equal, and they will be found to be at the maximum at the support. The distribution of the shearing stress over the area of the cross section is explained in the next chapter and need not be considered here. The amount of shearing stress is not due to any question of leverage, but is due to the amount and nature of the loading, and not to the span, as in the case of the bending moment. The conditions under which it is likely to become a serious consideration are those where a heavy load has to be carried over a short span, as in this case the bending moment will be comparatively small the leverage being small.

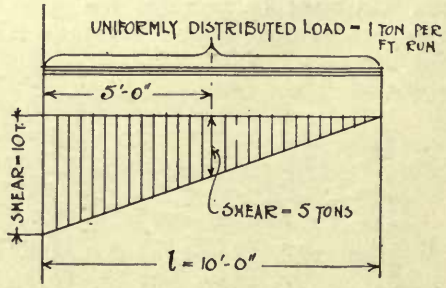


Fig. 112.—Shearing Stress in Cantilever which carries Uniformly Distributed Load

The possibility of failure by shear is frequently ignored on account of the fact that the amount of material required to resist the bending moment is usually such that ample provision for shear is given. In the case of heavy loads over a short span this does not always follow, and consequently the shearing stress must be calculated for as well as the bending moment. With regard to the distribution of the shearing stress over the length of the cantilever or beam, it must be fully realised at the outset that this stress is due to the opposition of the load and reaction, and consequently the value of the shear at any point can only be equal to the load that is travelling through that point to reach the abutment.

As a first example, consider a cantilever that is carrying a concentrated load at its outer end. Then at the support the shearing stress will be equal to the reaction, as at this point the total load will be opposed by the total reaction, and the opposition is therefore at the maximum. If any intermediate point in the cantilever is considered, it will be seen that the shear is still equal to

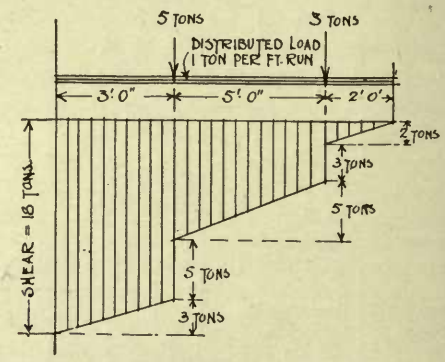


Fig. 113.—Shearing Stress in Cantilever which carries Uniformly Distributed Load and Two Concentrated Loads

the total load, as in passing outward from the support no load is passed over or left behind, as it were, until the extreme outer end is reached. The shearing stress at all points will therefore be as indicated in Fig. 111, where the length of the vertical line at any point equals the shear at that point. If W = 10 tons, then the shearing stress at the support or any intermediate point equals 10 tons.

In the case of a cantilever carrying a uniformly distributed load, the shearing stress at all points is somewhat different, as will be seen on reference to Fig. 112. In this case the shear will uniformly diminish from the support to the outer end, where it will be nil. In passing out from the support, it will be obvious that the load will be gradually passed over, as, for example,

at a point 1 ft. from the support 1 ton will be left behind and the opposition of 10 tons reduced by this amount, giving a shearing stress equal to 10 − 1 = 9 tons. When 5 ft. has been passed over, then 5 tons will be left between this point and the support, and the shear will equal 10 tons − 5 tons = 5 tons, as shown in the diagram.

It has been stated that the shear at any point can only equal the load that is passing through that point on its way to the support, or, in other words, *in a cantilever the shear at any point is equal to the load between that point and the other end.* Apply this rule to a cantilever carrying both distributed and concentrated loads; it will be shown how the shear is obtained. The example in Fig. 113 shows a cantilever carrying a uniformly distributed load of 1 ton per foot run over a length of 10 ft., together with

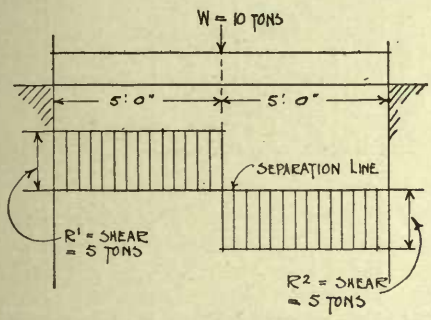


Fig. 114.—Shearing Stress in Beam which carries a Central Concentrated Load

two concentrated loads of 3 and 5 tons respectively. Beginning at the outer end and passing inward, and applying the rule that the shear at any point equals the load between that point and the outer end, we shall have placed a distributed load of 2 tons between us and the outer end by the time we have passed over 2 ft., and consequently there will be a gradually increasing shear from nil to 2 tons; and upon passing over the 2 ft. mark the centre of gravity of the concentrated load of 3 tons will be passed and placed between us and the outer end, and the shear is therefore suddenly increased by 3 tons, giving a total of 5 tons. The next 5 feet give a gradually increasing shear of 5 tons, which must be set downwards on the diagram, and immediately on passing the line of the second concentrated load another 5 tons is added, giving a shear of 15 tons. From this point to the support we pass over a distance of 3 ft., and gradually leave behind the remaining portion of the distributed load, equal to 3 tons, thus giving a total shear at the support of 18 tons, which is equal to the reaction at the support. This is clearly illustrated on the diagram referred to. The same principles and rule apply to any kind of loading, whether partly distributed or partly concentrated, bearing in mind that a distributed load causes a varying shear, represented by a sloping line, and a concentrated load causes a sudden increase in the shear equal to the amount of the load.

Beams require a little more consideration on account of there being two supports, and consequently a portion of the loading travels one way and is opposed by one reaction and the remainder travels in the opposite direction and is opposed by the other reaction. We have, therefore, a shear towards the left-hand abutment, caused by the opposition of R^1, and a shear toward the right-hand abutment caused by R^2, and the shear at either support will equal the reaction at that support. In the case of a beam carrying a central concentrated load only, as illustrated in Fig. 114, the reactions will each be equal to $\frac{W}{2}$, which in this case is $\frac{10}{2} = 5$ tons. To distinguish between the shear towards the left-hand and right-hand abutments, a horizontal line is drawn, known as the separation line, and the value of R^1 is set up to scale over this line and the value of R^2 below as shown. The full opposition will exist at all points between the support and the centre of gravity of the load, as the whole of the load which travels to the support will be passing through every point in the beam, and there is consequently no diminution in the shear at any point.

The rule for the shear at any point in a beam can be expressed as follows: *the shearing stress at any point in a beam is equal to the reaction at the support minus any load situated between that point and the support selected.* If the load between the point and support selected is greater than the reaction, then it indicates a shear toward the other support. As an illustration of this, supposing the shear at a point 7 ft. 6 in. from the left-hand abutment shown in Fig. 114 is to be calculated from R^1. Then (R^1 − the load of 10 tons) = (5 tons − 10 tons) = −5 tons, thus indicating a shear toward

the right-hand abutment which agrees with the diagram. In the case of a uniformly distributed load on a beam, the shear at either abutment will be equal to the reaction at that abutment, and it will gradually diminish to the centre, where it will be *nil*, as illustrated in Fig. 115. An example of combined concentrated and distributed loading is given in Fig. 116, and if this is explained no difficulty should be encountered by the student in working out other examples, however complicated the loading may be.

As the reaction at the support is equal to the shear, it will be necessary to calculate this in the first instance. Therefore R^1 will equal half the distributed load $= \frac{10}{2} =$ 5 tons $+ \frac{(3 \times 8) + (4 \times 6) + (5 \times 3)}{10} =$

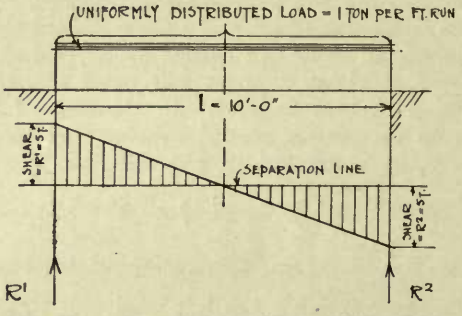


Fig. 115.—Shearing Stress in Beam which carries Uniformly Distributed Load

5 tons $+ \frac{24 + 24 + 15}{10} =$ 5 tons $+ 6\frac{3}{10}$ tons $= 11\frac{3}{10}$ tons. Working out $R^2 =$ 5 tons $+ \frac{(3 \times 2) + (4 \times 4) + (5 \times 7)}{10} =$ 5 tons $+ \frac{6 + 16 + 35}{10} =$ 5 tons $+ 5\frac{7}{10} = 10\frac{7}{10}$ tons. $R^1 + R^2 =$ total loading, therefore $11\frac{3}{10} + 10\frac{7}{10} = 22$ tons, which is correct. If we now draw a horizontal separation line and set up R^1 to scale at the left-hand abutment, we shall have the shear at this point, and we can proceed to find the shear at all points. After passing over the first 2 feet we have reduced the shear by 2 tons, giving $9\frac{3}{10}$ tons at this point, and directly we pass this mark we reduce the stress by 3 tons, due to the concentrated load leaving a shear of $6\frac{3}{10}$ tons. We now gradually leave behind a further 2 tons by the time the next concentrated load is reached, making a reduction to $4\frac{3}{10}$ tons, and upon passing the concentrated load we only have $\frac{3}{10}$ ton left as shear toward the left-hand abutment. As the distributed load equals 1 ton per foot run, it will be obvious that when we have passed over a further distance of $\frac{3}{10}$ ft., we shall have no shear to this abutment, and it is therefore at this point that the separation line will be crossed and the shearing stress will be below the line, and thus towards the right-hand abutment. This point of crossing the line will

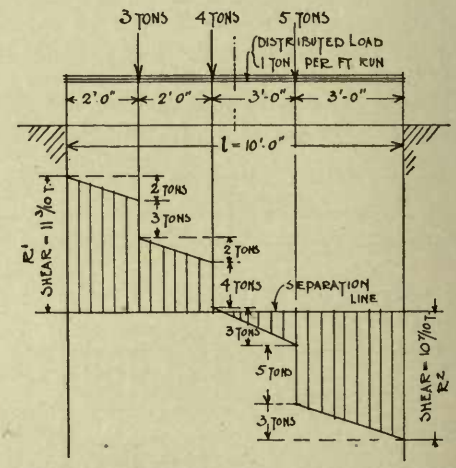


Fig. 116.—Shearing Stress in Beam which carries Uniformly Distributed Load and Three Concentrated Loads

be $4\frac{3}{10}$ ft. from the left-hand abutment, and it is important to be able to calculate this in the case of reinforced concrete beams when special shear members are provided as shown in the notes on this work. In continuing along the beam it will be seen that after passing the point of crossing the separation line a distance of $2\frac{7}{10}$ ft. will be passed over before reaching the load of 5 tons, thus picking up $2\frac{7}{10}$ tons, and upon adding the 5 tons the shear will equal $7\frac{7}{10}$ tons. The remaining 3 ft. will add a gradually increasing shear of 3 tons, making a total of $10\frac{7}{10}$ tons upon reaching the support, and this amount is equal to the reaction, and therefore is correct. Fig. 116 shows the shear at all points in a graphic manner, and should assist the student in following the explanation.

The Theory of Reinforced Concrete

Introductory.—The theory of reinforced concrete is, unfortunately, in a very unsatisfactory state at the present time, and, in fact, it may be said that no kind of universal agreement has been arrived at. Students have no definite course of study that can be followed with the surety that diligent application will enable them to design and calculate the material in a manner consistent with a universally recognised standard. The desirability of standardising the methods of design and calculation is fully recognised by the various Institutes who are using every endeavour to bring about this reform; and, in the meanwhile, it is incumbent upon all writers on the subject to help matters forward by adopting the recommendations of the leading Institutes as regards notation and general methods.

For the purpose of this treatise, it has been decided to adopt the notation recommended by the Concrete Institute, this being regarded by the writer as the most satisfactory that has been compiled. The working stresses employed are those contained in the second report of the Royal Institute of British Architects.

A knowledge of stresses and strains is necessary to every student before an attempt is made to take up the calculations for reinforced concrete. Therefore, although these notes apply to simple theory only, it is necessary to assume that the reader has mastered the preceding chapter and possesses the knowledge referred to; and only those explanations will be given which are essential to a proper understanding of the methods of deduction. There is a great tendency to render formulæ and explanations very complicated, with the result that many students possessing only a limited knowledge of mathematics are unable to follow them. Bearing this in mind, the writer has endeavoured to delete all unnecessary complications and to put the matter in a simple manner, which is capable of being understood by all readers. The many different symbols used in the formulæ are apt to look formidable at first sight, but the reader is advised to make every effort to become familiar with them and to endeavour to realise the full meaning of each one.

It is not sufficient to learn several formulæ and to be able to put them down from memory, as this will not enable them to be used intelligently; each one must be understood and its construction realised. Indeed, the user should be able to construct his own formulæ when he has fully grasped the principles of force and resistance.

The calculations can be divided into two sections, namely, (1) the determination of the value of the external force or forces, and (2) the determination of the necessary resistance values to overcome these forces.

The value of the force to cause stress will depend upon the nature, amount, and disposition to the resisting member, and the value of the resistance will depend upon the shape and size of the member and the material of which it is composed.

DATA FOR CALCULATIONS

Ascertaining Loads to be Carried.—There must be some basis upon which to found the calculations, and the following methods of ascertaining the loads to be carried must be taken into consideration:

(1) The weight of the structure, including flooring, plaster, and any applied decoration or similar loading.

(2) The superimposed or accidental load, which will vary in amount according to the class of building being designed.

(3) Vibration and shock wherever this is likely to occur. This will vary with the class of building, as obviously a greater allowance must be made in the case of a building containing machinery than in the case of an ordinary dwelling house.

In the case of item No. 1 the weight of the concrete and steel may be taken at 150 lb. per foot cube. To this must be added the flooring material or any other weight, according to circumstances.

The load per square foot to be allowed, as in item 2, may be taken on the following basis:—

	lb. per sq. ft.
Ordinary dwelling house, flats, hotel bedrooms, and hospital wards	75
Offices, schoolrooms, etc. . .	100
Theatres, libraries, concert-rooms, banks, and shops . . .	120
Ballrooms and drill halls . .	150
Warehouses and stores vary according to use . . .	230–670
Factories and machine shops. These will vary greatly, according to the number and weight of machines, and each case must have special consideration.	
Roofs generally	50

The allowance for shock mentioned in item 3 will be necessary in the case of buildings which are subjected to greatly varying loads, as, for instance, public buildings, factories, or workshops, and such allowance in these cases should be taken at one half the superimposed load as given above for the various buildings. In structures where the vibration is caused by machinery or heavy traffic, as in the case of vault roofs, the allowance should be equal to the actual superimposed load.

In the case of columns and piers which carry several stories above, a certain reduction of the load picked up at the different levels is permissible, as follows:—For the part of the roof or top floor supported, the actual superimposed load calculated is taken. For the next floor below a reduction of 10 per cent. is made, and for the floor below this a reduction of 20 per cent. is made, and so on until the floor is reached at which the reduction is equal to 50 per cent. Below this level all the floors are taken at 50 per cent. below the superimposed loads in determining the reactions on the columns.

This reduction must, of course, be used with discretion, as it is based on the assumption that all the floors would not be loaded to their full capacity at the same time, and therefore, in the case of warehouses and some buildings where all the floors may be heavily loaded together, it is wise to make no reduction whatever.

Working Stresses.—The following working stresses are based on the assumption that the concrete is of such a quality that its crushing strength is at least 1,800 lb. per sq. in. after twenty-eight days, and that the steel has a tensile strength of not less than 60,000 lb. per sq. in.:—

	lb. per sq. in.
Concrete in compression in beams subjected to bending . .	600
Concrete in columns under simple compression	600
Concrete in shear in beams .	60
Adhesion or grip of concrete to metal	100
Steel in tension	16,000
Steel in compression—fifteen times the stress in the surrounding concrete	
Steel in shear	12,000

The resistance of concrete to tension is neglected, as its value is small, and, in addition, it cannot be considered as a reliable material to take such a stress; therefore, the steel is calculated to take all the tensile stress. The figures given above for concrete are based on tests made with concrete made in the proportions of 1 cement, 2 sand, and 4 hard stone, and should different proportions be adopted the stress may be taken at one-third the crushing strength twenty-eight days after moulding. This may be considered a low factor of safety by some designers, but it must be borne in mind that the maximum strength of the material is not developed in twenty-eight days.

If the steel has a greater ultimate strength in tension than 60,000 lb. per sq. in., the safe stress may be taken at one half of the stress required to reach the elastic limit of the material; but in no case is it advisable to calculate for a greater stress than 20,000 lb. per sq. in.

The stress is considered as acting on the concrete in a uniformly varying manner, while in the case of the steel it is considered as uniform over the cross section.

Elasticity.—The question of the relative elasticity of the two materials is highly important, and although there is some difference of opinion on this point, it is now generally accepted that the coefficient of elasticity for concrete in compression gauged as above mentioned is constant and can be taken at one-fifteenth that of steel. The modulus for concrete is expressed by E_c, and equals 2,000,000 lb. per sq. in. The modulus for steel is expressed by E_s, and equals 30,000,000 lb. per sq. in.; therefore $\frac{E_s}{E_c} = 15$. It is highly important that the

student should realise this point, as it means that at any given distance from the neutral axis the stress per square inch on the steel will be fifteen times as great as that on the concrete. In other words, the stress can only be proportional to the resistance where equilibrium is produced, and as the steel is capable of offering fifteen times as much resistance to stretching as the concrete is to compression, it will require fifteen times more stress in the steel to produce the same effect as that taking place in the concrete.

Spans.—The span to be considered in the calculations will not, of course, be the clear span, but the effective span, and this, in the case of beams, may be taken as the distance between the centres of bearings. For slabs supported at the ends only, the distance will be the clear span plus the thickness of the slab; while for continuous slabs it becomes the distance from centre to centre of the supporting beams.

Bending Moments.—The question of bending moments has been fully dealt with in the preceding chapter, but it must be borne in mind that a slab can only be considered as continuous when it is continued over three or more spans, and then the spans must be equal and the supports at the same level. With a uniformly distributed load the maximum bending moment may be taken as $+\frac{Wl}{12}$ at the centre and $-\frac{Wl}{10}$ at the intermediate supports, where W = total weight and l equals span.

BEAMS WITH SINGLE REINFORCEMENT

Some little difficulty is experienced by the student in determining the moment of resistance of a reinforced beam, as there are two materials to be considered, each having a different modulus of elasticity and resistance to stress. It is necessary, however, to be able to determine their moment of resistance at any point in order that it may be shown that it equates with the bending moment at the same point.

The bending moment represents the value of the external forces to destroy the beam by tension or compression, the former taking place on the lower part of the beam and the latter on the upper, where the beam is supported at the ends only. If the beam is fixed at the ends, then the beam will be called upon to resist compression in the lower part and tension in the upper part for a certain distance from the fixed ends, but for the purpose of this present explanation it will be considered that the beam is supported only.

There are several formulæ for the moment of resistance of a reinforced concrete beam, but it is not sufficient to give these without explaining the principle of their construction.

It is shown in the preceding chapter that the compressional and tensional resistances must be equal to one another, and that they are acting in opposite directions and are parallel, thus forming a "couple," and the value of the moment of resistance of the whole section is the amount of one resisting power multiplied by its distance from the other.

The resistance to compression is supplied by the concrete above the neutral axis, while the resistance to tension is supplied by the steel in the bottom of the beam, the concrete below the neutral axis being neglected, as shown in Figs. 117 and 118. Here it will be seen that the stress on the concrete is shown as being of a uniform intensity acting over an equivalent inertia area which, as previously explained, is equal to a varying intensity over the whole of the compressional area. Hence the equivalent compressional area will equal $\frac{bn}{2}$, where b = the breadth of the beam in inches and n = the distance of the neutral axis from the compressed edge. The safe compressional stress per square inch on the concrete is expressed by c, and the value of the compressional resistance, therefore $= \frac{cbn}{2}$. The safe resistance of the steel in tension per square inch is expressed by the symbol t, and the sectional area in square inches by A_t, therefore the value of the tensional resistance equals $A_t \times t$.

The total compression and total tension are equal, and therefore $\frac{cbn}{2} = tA_t$. These two equal resistances or internal forces act in opposite directions, and the amount of one of them must be multiplied by the distance between them to find the moment of resistance. The centre of gravity of the compressional area, at which point the compressional resistance can be considered as being concentrated, will be situated at a distance of $\frac{n}{3}$ from the top edge, or at a distance of $\frac{2}{3}n$ from the neutral axis. The tensional resistance will be acting at a

point equal to $d - n$ from the neutral axis or $d - \frac{n}{3}$ from the centre of gravity of the compressional area. Therefore the moment of resistance $= \frac{c\,b\,n}{2} \times \left(d - \frac{n}{3}\right)$ or $t\,A_t \times \left(d - \frac{n}{3}\right)$, and this must be equal to the bending moment, B M. Thus $\text{B M} = \frac{c\,b\,n}{2} \times \left(d - \frac{n}{3}\right)$ or $\text{B M} = t\,A_t \times \left(d - \frac{n}{3}\right)$.

Before any use can be made of these formulæ it will be necessary to establish some relation between certain factors which are variable and unknown, namely, c, t, and n.

The factors b, d, and A_t can be settled by the designer himself, but the value of n, t, and c will be affected by the relation or proportion of A_t to $b\,d$, and as there are two unknown factors in each formula it would not be possible to proceed beyond a certain point.

Again, it will not be possible to stress both the steel and the concrete to the permissible limit unless they are in the correct proportion to one another. The two materials have a different coefficient of elasticity which in the concrete is expressed by E_c and in the steel by E_s, and the ratio between them $= \frac{E_s}{E_c} = 15$, and this is an important point in the determination of the stresses. It signifies that in the case of deformation taking place where the concrete is compressed by a certain stated amount (say, 1 in.), while the steel is stretched by the same amount, it will require fifteen times as much stress on the steel as that on the concrete to produce this result if the two materials are situated at the same distance from the neutral axis. This is illustrated in Fig. 119, where it is assumed for the moment that the neutral axis is situated at the centre of the depth; then the permissible stress on the concrete is 600 lb. per sq. in., and the stress on the steel will equal $c \times m$, which equals $600 \times 15 = 9{,}000$ lb., m being the ratio between E_s and E_c.

It must be clearly understood that although the deformation does not actually take place if the beam is properly designed, there is a tendency for it to do so, and consequently the resistance has to be exerted by the materials to overcome the tendency.

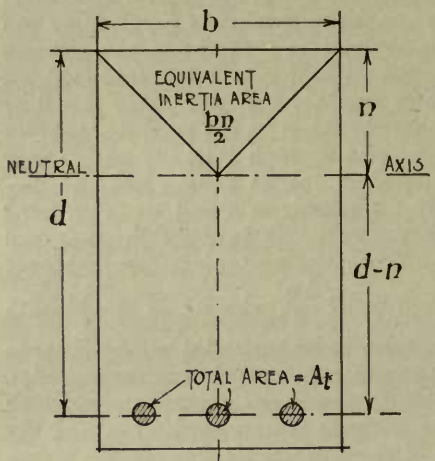


Fig. 117

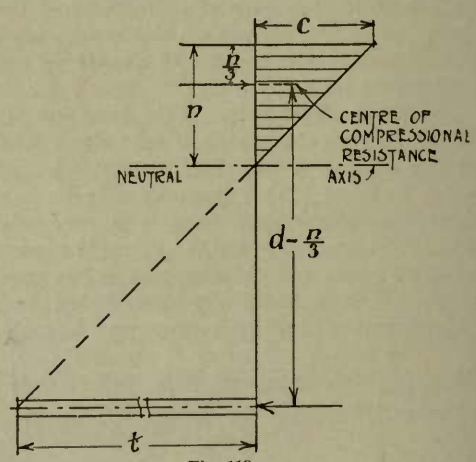


Fig. 118

Figs. 117 and 118.—How a Single Reinforced Beam Resists Compression and Tension

The permissible stress on the steel is 16,000 lb., and therefore if the proportion between the two materials is such that the neutral axis is at the centre of the depth, the steel cannot be used to its fullest capacity. It will be necessary to have the proportion such that the neutral axis is higher up the beam, and thus farther from the steel, because the stress varies directly as the distance from the neutral axis, and thus, in addition to the ratio m, it is necessary to consider the relative distances n and $d - n$. If the steel is 2 in. from the neutral axis while the concrete is only 1 in. from the same point, then the stress on the steel will be equal to twice that in the concrete; or, in other words, $t = c \times 2$. The same rule will apply if it is said that the stress in the steel multiplied by n will equal the stress

in the concrete multiplied by $d - n$, or $t \times n = c \times d - n$. Now taking into consideration the ratio m and the distance from the neutral axis, the result is $t\,n = m\,c\,(d - n)$. It is allowable to reduce this to give a relative proportion for n as follows:—

$$t\,n = m\,c\,(d - n)$$
$$t\,n = m\,c\,d - m\,c\,n$$
$$t\,n + m\,c\,n = m\,c\,d$$
$$n\,(t + m\,c) = m\,c\,d$$
$$n = \frac{m\,c\,d}{t + m\,c} \text{ or } \frac{n}{d} = \frac{m\,c}{t + m\,c}$$

In the case of this formula there are certain factors known if c and t are considered as representing the permissible stress on the concrete and steel; thus, $m = 15$, $c = 600$ lb.,

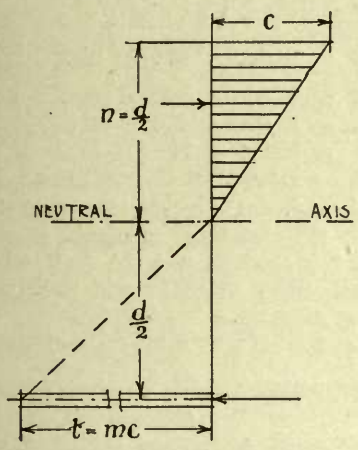


Fig. 119. — Diagram Showing Proportionate Stresses above and below Neutral Axis to Produce Deformation

and $t = 16{,}000$ lb. With the aid of these it is possible to deduce n as a definite proportion of d, as follows:—

$$n = \frac{m\,c\,d}{t + m\,c}$$
$$n = \frac{15 \times 600 \text{ lb.} \times d}{16{,}000 \text{ lb.} + (15 \times 600 \text{ lb.})}$$
$$n = \frac{9{,}000\,d}{25{,}000}$$
$$n = \frac{9}{25}\,d = n = \cdot 36\,d.$$

Now this is of great value, as it shows that in the case of a well-designed beam where t and c are at the permissible limit, then $n = \cdot 36\,d$. It does not necessarily follow that c and t will be equal to the values given above, as it has previously been stated that this will only occur when A_t and $b\,d$, the areas of the steel and concrete, bear a certain proportion to one another.

The next step, then, is to determine this proportion, and see how the neutral axis will vary if another proportion is employed.

The ratio of the area of the tensile reinforcement to the area $b\,d$ is expressed by the symbol r, therefore $r = \frac{A_t}{b\,d}$.

Now it has already been seen that $t\,A_t = \frac{c\,b\,n}{2}$, so that $A_t = \frac{c\,b\,n}{2\,t}$. If $r = \frac{A_t}{b\,d}$ then r also equals $\frac{c\,b\,n}{2\,t\,b\,d}$, as $\frac{c\,b\,n}{2\,t}$ is the equivalent of A_t and can be substituted for it. In the expression $r = \frac{c\,b\,n}{2\,t\,b\,d}$, the symbol b can be eliminated, as it occurs in both cases, thus reducing it to $r = \frac{c\,n}{2\,t\,d}$.

An expression is now obtained which will give the ratio of the steel to the concrete to develop certain given values for c and t. In the first place, consider the case where $c = 600$ lb. and $t = 16{,}000$ lb. as above, where it was seen that $n = \cdot 36\,d$, then c, n, t, and d are known, thus—

$$r = \frac{600 \times \cdot 36}{2 \times 16{,}000 \times 1}$$
$$r = \frac{216}{32{,}000}$$
$$r = \frac{27}{4{,}000} = \cdot 00675.$$

It will be seen by this that if the greatest permissible stress is represented by $c = 600$ lb. and $t = 16{,}000$ lb., then $n = \cdot 36\,d$ and $r = \cdot 00675$ or $A_t = \cdot 00675\,b\,d$.

Applying the Formulæ.—Before proceeding to the derivation of any more formulæ, it will be as well to apply those obtained up to the present and become more or less familiar with their use, to prevent the subject from becoming too complicated for the student who is dealing with symbols and reasoning which are quite new to him.

Briefly summarised, the following have been deduced: (1) The total compression and total tension are equal, expressed $\frac{c\,b\,n}{2} = t\,A_t$. (2) The moment of resistance equals either of these multiplied by $d - \frac{n}{3}$, which is the lever arm of the internal forces,

expressed $M R = \frac{c b n}{2} \times \left(d - \frac{n}{3}\right)$, or $M R = t A_t \times \left(d - \frac{n}{3}\right)$. (3) These must be equal to the bending moment, expressed $B M = \frac{c b n}{2} \times \left(d - \frac{n}{3}\right)$ or $B M = t A_t \times \left(d - \frac{n}{3}\right)$. (4) The position of the neutral axis is dependent on the coefficients of elasticity, $m = 15$, and the values of c and t, expressed $n = \frac{m c d}{t + m c}$, where $c = 600$ lb. and $t = 16{,}000$ lb., then $n = \cdot 36\, d$. (5) The ratio of the steel reinforcement to the area of the beam will depend on the values of c and t, and will be expressed $r = \frac{c n}{2 t d}$, when $c = 600$ lb. and $t = 16{,}000$ lb.; then $r = \cdot 00675$ or $A_t = \cdot 00675\, b\, d$.

Calculating Amount of Reinforcements.—As a first example, assume a concrete beam 12½ in. deep and 8 in. wide. It is required to find the amount of steel necessary to reinforce the beam if the working stresses are to be $c = 600$ lb. and $t = 16{,}000$ lb. per sq. in. Also calculate the moment of resistance of the beam when so reinforced, and find what uniformly distributed load it will carry over a span of 15 ft. if the ends are supported only.

The values of c and t being known, the first step is to ascertain the position of the neutral axis. By using the formula in deduction No. 4, it will be found $n = \cdot 36\, d$.

Therefore $n = \cdot 36 \times 12{\cdot}5$

$n = 4{\cdot}5$ in.

Work this out by means of the formula $n = \frac{m c d}{t + m c}$ for the purpose of getting quite familiar with the use of same, then $n = \frac{15 \times 600 \times 12{\cdot}5}{16{,}000 + 15 \times 600}$ $n = \frac{112{,}500}{25{,}000} = n = 4{\cdot}5$ in., as before.

Having derived the value of n, the amount of steel will be given by $r = \frac{c n}{2 t d}$, and this should work out at ·00675, as $c = 600$ lb. and $t = 16{,}000$ lb.

$$r = \frac{600 \times 4{\cdot}5}{2 \times 16{,}000 \times 12{\cdot}5}$$

$$r = \frac{2{,}700}{400{,}000}$$

$$r = \cdot 00675$$

A_t therefore equals $\cdot 00675\, b\, d$.

$$A_t = \cdot 00675 \times 8 \times 12{\cdot}5$$

$$A_t = \cdot 00675 \times 100$$

$$A_t = \cdot 675 \text{ sq. in.}$$

The moment of resistance can now be calculated according to formulæ in deduction No. 2, namely:—

$$M R = \frac{c b n}{2} \times \left(d - \frac{n}{3}\right)$$

$$M R = \frac{600 \times 8 \times 4{\cdot}5}{2} \times \left(12{\cdot}5 - \frac{4{\cdot}5}{3}\right)$$

$$M R = \frac{21{,}600}{2} \times (12{\cdot}5 - 1{\cdot}5)$$

$$M R = 10{,}800 \times 11$$

$$M R = 118{,}800 \text{ in.-lb.}$$

Again, working the moment of resistance according to the tensile reinforcement,

$$M R = t A_t \times \left(d - \frac{n}{3}\right)$$

$$M R = 16{,}000 \times \cdot 675 \times \left(12{\cdot}5 - \frac{4{\cdot}5}{3}\right)$$

$$M R = 16{,}000 \times \cdot 675 \times (12{\cdot}5 - 1{\cdot}5)$$

$$M R = 16{,}000 \times \cdot 675 \times 11$$

$$M R = 10{,}800 \times 11$$

$$M R = 118{,}800 \text{ in.-lb., as before.}$$

As the moment of resistance equals 118,800 in.-lb., the bending moment must not exceed this amount, and the next step is to ascertain what weight would produce this bending moment.

In the case of a uniformly distributed load with supported ends $B M = \frac{w l}{8}$, therefore 118,800 in.-lb. $= \frac{w l}{8}$. w is the unknown quantity, and l = the span in inches $= 15 \times 12 = 180$.

$$118{,}800 \text{ in.-lb.} = \frac{w \times 180}{8}$$

$$w = \frac{118{,}800 \times 8}{180}$$

$$w = 5{,}280 \text{ lb.}$$

$$w = 47{\cdot}14 \text{ cwt.}$$

The beam will therefore require ·675 sq. in. of steel reinforcement, and safely carry a load of 47·14 cwt. over a span of 15 ft.

Calculating Beam for Certain Load.—In this example, the size of the beam was given and the reinforcement and the load were calculated, for the sake of applying the deductions previously made. In actual practice it is usually required to design a beam to carry a certain load or to check a given beam and ascertain if the safe stresses on the concrete and steel have been exceeded. As an example of the latter

instance, the beam just dealt with can be calculated, when the stresses should agree with those arranged in the question, namely $c = 600$ lb. and $t = 16{,}000$ lb. The bending moment to be dealt with is 118,000 in.-lb., and as shown in deduction No. 3, $\text{B M} = \frac{c\,b\,n}{2} \times \left(d - \frac{n}{3}\right)$. The value of c will therefore be found by the formula $c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right)}$.

$$c = \frac{2 \times 118{,}800}{8 \times 4{\cdot}5 \times \left(12{\cdot}5 - \frac{4{\cdot}5}{3}\right)}$$

$$c = \frac{237{,}600}{396} = c = 600 \text{ lb.}$$

This amount is correct, and the stress in the steel can next be calculated as follows:—

$$\text{B M} = t\,\text{A}_t \times \left(d - \frac{n}{3}\right), \text{ therefore}$$

$$t = \frac{\text{B M}}{\text{A}_t \times \left(d - \frac{n}{3}\right)}$$

$$t = \frac{118{,}800}{{\cdot}675 \times \left(12{\cdot}5 - \frac{4{\cdot}5}{3}\right)}$$

$$t = \frac{118{,}800}{7{\cdot}425} = t = 16{,}000 \text{ lb.}$$

This, again, agrees with the stress allowed, and thus the two formulæ are obtained which will give the stresses in the beam when this has to be checked after it has been designed.

In the actual designing of the beam, the load will be a definite quantity according to circumstances, and the bending moment will therefore require to be calculated in the first instance. It will be necessary, therefore, to have some equation which will give a suitable size beam to resist such bending moment, and also some method of obtaining the position of the neutral axis when the values of c and t are unknown, such as would be the case in designing to resist a given bending moment.

The next step is to establish some relation between b, d, and the bending moment. The draft regulations of the London County Council limit the minimum depth of a beam to one-twenty-fourth of the span, but as the compressional resistance depends on the area of concrete above the neutral axis and the resistance must be equal to the bending moment, it is preferable to calculate from the bending moment direct without regard to the span. Referring to deduction No. 3, begin again with the equation $\text{B M} = \frac{c\,b\,n}{2} \times \left(d - \frac{n}{3}\right)$. Thus $2\,\text{B M} = c \times b \times n \times \left(d - \frac{n}{3}\right)$. The value of c can be taken at 600 lb., and n should equal ·36 d if the economical percentage of reinforcement is to be employed, this leaving only two unknown factors b and d. If b is settled as some definite proportion of d, then this proportion can be substituted and d expressed as a fraction of B M. A well-proportioned beam is frequently that in which $b = {\cdot}6\,d$, and this will be taken as the value, hence—

$$2\,\text{B M} = c \times b \times n \times \left(d - \frac{n}{3}\right)$$

$$2\,\text{B M} = 600 \times {\cdot}6\,d \times {\cdot}36\,d \times \left(d - \frac{{\cdot}36d}{3}\right)$$

$$2\,\text{B M} = 600 \times {\cdot}6\,d \times {\cdot}36\,d \times (d - {\cdot}12\,d)$$

$$2\,\text{B M} = 600 \times {\cdot}6\,d \times {\cdot}36\,d \times {\cdot}88\,d$$

$$2\,\text{B M} = 600 \times {\cdot}19008\,d^3$$

$$2\,\text{B M} = 114{\cdot}048\,d^3$$

$$\text{B M} = 57{\cdot}024\,d^3$$

$$d^3 = \frac{\text{B M}}{57{\cdot}024}$$

$$d = \sqrt[3]{\frac{\text{B M}}{57{\cdot}024}}$$

To simplify matters and save a great number of figures in the calculations, it will be sufficient to use $d = \sqrt[3]{\frac{\text{B M}}{57}}$, the deletion of the decimal figures in this case making so little difference that it is inappreciable. The writer is aware that there are many students who have forgotten the method of finding the cube root of a number, but when this is the case the student should use a table and thus overcome any difficulty on this point until such time as he shall again become conversant with the method. In the absence of a table, the cube root can be found by trial, but, needless to mention, this is somewhat of a tedious method.

To illustrate the application of the formula, it will be advisable to work out an example, as before, the load and span being given. It is required to design a beam supported at the ends to carry a load of 650 lb. per foot run over an effective span of 15 ft. It is assumed that this load includes the weight of the beam itself, and the safe stresses are to be 600 lb. and 16,000 lb.

The total load $= 650$ lb. $\times 15 = 9{,}750$ lb.

The bending moment $= \frac{w\,l}{8} = \frac{9750 \times 180}{8}$

$$\text{B M} = 219{,}375 \text{ in.-lb.}$$

Therefore $d = \sqrt[3]{\frac{219375}{57}}$

$$d = \sqrt[3]{3848}$$

$$d = 15{\cdot}7 \text{ in.}$$

The breadth should be $\cdot 6\,d$, therefore

$$b = \cdot 6 \times 15{\cdot}7 \text{ in.}$$

$$b = 9{\cdot}4 \text{ in.}$$

The size of the concrete beam itself has therefore been found, and the amount of reinforcement is now required.

According to deduction No. 5, the ratio of steel to concrete should be ·00675. Therefore $A_t = \cdot 00675\,b\,d$, $b\,d = 147{\cdot}58$ sq. in., and $A_t = \cdot 00675 \times 147{\cdot}58$. $A_t = \cdot 996$ sq. in. —say three $\frac{11}{16}$-in. diameter round bars. This will give the nearest area, always bearing in mind that there should be an excess rather than a deficiency. The area of one $\frac{11}{16}$-in. diameter bar $= \cdot 3712$ sq. in., and the total area of the three bars $= 1{\cdot}1136$ sq. in.

The beam as designed has an effective depth of 15·7 in. To this must be added, say, 2 in. of concrete to cover the bars, giving a total depth of 17·7 in. The completed section should therefore be: depth, $17\frac{3}{4}$ in.; breadth, $9\frac{1}{2}$ in.; reinforcement, No. 3 $\frac{11}{16}$-in. diameter round rods.

Checking the Beam.—In the case of beams which have to be checked for the purpose of ascertaining the values of c and t, the necessary formulæ up to a certain point are at hand, but some further explanations and deductions are required to make the data complete. As previously shown, the stress in the concrete can be found by the formula $c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right)}$, and the stress in the steel can be ascertained by $t = \frac{\text{B M}}{A_t \times \left(d - \frac{n}{3}\right)}$. The difficulty in the case of these formulæ will be that of finding the value of n. Previously, the position of n has been said to be dependent on the coefficients of elasticity and the values of c and t.

In the case of checking a beam already designed c and t are unknown, and the formula cannot be used, and if the ratio of steel to concrete is not ·00675, then n will not equal $\cdot 36\,d$. The position of the neutral axis, being dependent upon the comparative values of c and t, must also be dependent upon the proportion of steel to concrete, and as both of these will be given in the beam as designed, a value for n must be deduced accordingly.

It has already been shown that $t\,n = c\,m\,(d - n)$, and therefore $\frac{t}{c} = \frac{m \times (d - n)}{n}$.

Again, $\frac{c\,b\,n}{2} = t\,A_t$, and therefore $n = \frac{2\,t\,A_t}{c\,b}$. Now t and c are unknown, in which case substitute the value for $\frac{t}{c}$, namely, $\frac{m\,(d - n)}{n}$. Then $n = \frac{2\,A_t\,m\,(d - n)}{b\,n}$, and $b\,n^2 = 2\,A_t\,m\,(d - n)$. Here n is on both sides of the equation, and it must be reduced as follows:—

$$b\,n^2 = 2\,A_t\,m\,(d - n)$$

$$b\,n^2 = 2\,A_t\,m\,d - 2\,A_t\,m\,n$$

$$b\,n^2 + 2\,A_t\,m\,n = 2\,A_t\,m\,d$$

Divide both sides by b, then

$$n^2 + \frac{2\,A_t\,m\,n}{b} = \frac{2\,A_t\,m\,d}{b}$$

Add $\left(\frac{A_t\,m}{b}\right)^2$ to both sides of the equation, then

$$n^2 + \frac{2\,A_t\,m\,n}{b} + \left(\frac{A_t\,m}{b}\right)^2 = \frac{2\,A_t\,m\,d}{b} + \left(\frac{A_t\,m}{b}\right)^2$$

$$\left(n + \frac{A_t\,m}{b}\right)^2 = \frac{2\,A_t\,m\,d}{b} + \left(\frac{A_t\,m}{b}\right)^2$$

$$n + \frac{A_t\,m}{b} = \sqrt{\frac{2\,A_t\,m\,d}{b} + \left(\frac{A_t\,m}{b}\right)^2}$$

Therefore

$$n = \sqrt{\frac{2\,A_t\,m\,d}{b} + \left(\frac{A^t\,m}{b}\right)^2} - \frac{A_t\,m}{b}.$$

Finding Position of Neutral Axis.—A formula is now to hand for finding the position of the neutral axis in the case of a beam which has been designed, where the stresses may or may not be in accordance with the permissible limits.

Assume, as a first example, that the beam given in Fig. 120 has been designed to resist a bending moment of 219,375 in.-lb., and it is required to ascertain if the stress in the concrete and steel is in accordance with the values allowed, namely 600 lb. and 16,000 lb. This is actually the same beam that was calculated in the previous example, and it should be noted that the area of the steel

reinforcement required was only ·996 sq. in.; but it was increased to 1·1136 to make it possible to use rods of a practical size, and it is for the latter area that it must now be calculated. The sizes of the concrete are also the practical sizes, such as would appear on a drawing of the beam, when the theoretical sizes would not be available to anyone checking the stresses in the ordinary way.

The first step is to ascertain the position

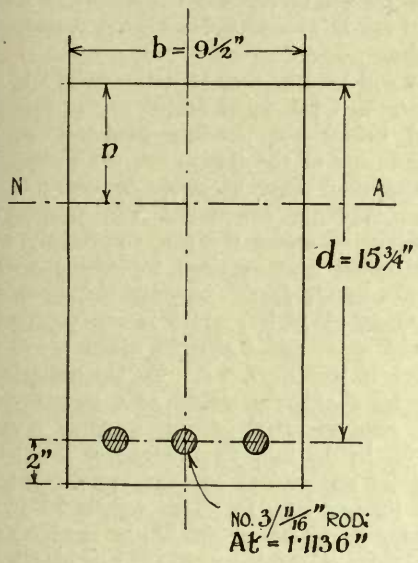


Fig. 120.—Finding Position of Neutral Axis

of the neutral axis, and this will be done by means of the formula given above, namely :—

$$n = \sqrt{\frac{2\,A_t\,m\,d}{b} + \left(\frac{A_t\,m}{b}\right)^2} - \frac{A_t\,m}{b}$$

$$n = \sqrt{\frac{2 \times 1{\cdot}1136 \times 15 \times 15{\cdot}75}{9{\cdot}5} + \left(\frac{1{\cdot}1136 \times 15}{9{\cdot}5}\right)^2} - \frac{1{\cdot}1136 \times 15}{9{\cdot}5}$$

$$n = \sqrt{\frac{526{\cdot}176}{9{\cdot}5} + \left(\frac{16{\cdot}704}{9{\cdot}5}\right)^2} - \frac{16{\cdot}704}{9{\cdot}5}$$

$$n = \sqrt{55{\cdot}386 + (1{\cdot}758)^2} - 1{\cdot}758$$

$$n = \sqrt{58{\cdot}476} - 1{\cdot}758$$

$$n = 7{\cdot}64 - 1{\cdot}758$$

$$n = 5{\cdot}882 \text{ in.}$$

It will be seen that this result differs slightly from that which would have been obtained by taking $n = {\cdot}36\,d$, as in the latter case this equals ·36 × 15·75 = 5·67 in., and this difference has arisen on account of A_t and $b\,d$ having been altered to give practical sizes. Having now obtained the value of n, the values of c and t can be found as follows :—

$$c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right)}$$

$$c = \frac{2 \times 219375}{9{\cdot}5 \times 5{\cdot}882 \times \left(15{\cdot}75 - \frac{5{\cdot}882}{3}\right)}$$

$$c = \frac{438750}{9{\cdot}5 \times 5{\cdot}882 \times 13{\cdot}70}$$

$$c = \frac{438750}{770{\cdot}57}$$

$$c = 569 \text{ lb.}$$

This is below the permissible stress of 600 lb. per sq. in., and the value of t may now be determined :—

$$t = \frac{\text{B M}}{A_t \times \left(d - \frac{n}{3}\right)}$$

$$t = \frac{219375}{1{\cdot}1136 \times \left(15{\cdot}75 - \frac{5{\cdot}882}{3}\right)}$$

$$t = \frac{219375}{1{\cdot}1136 \times 13{\cdot}79}$$

$$t = \frac{219375}{15{\cdot}356}$$

$$t = 14285 \text{ lb.}$$

This value, again, is well below the permissible stress, and consequently the beam is quite satisfactory for the load it has to carry.

It is interesting to note that the area of steel required theoretically was increased by about 12 per cent. to give a practical area, and this reduced the stress on the steel per square inch by about 10 per cent., and the stress on the concrete by about 5 per cent.

In the draft regulations of the London County Council, it will be seen that the formula for finding the position of the neutral axis is given as :

$$n = \left[\sqrt{m^2 r^2 + 2\,m\,r} - m\,r\right] d$$

and a note in explanation of this should be of interest to those who are not sufficiently advanced to trace the construction and evolution of the formula. The symbol r, as previously stated, equals the ratio of steel to concrete, and the formula is constructed to use this ratio for finding the position of n when c and t are unknown. Upon reference to the reasoning employed in deducing the previous formula for the position of n, it will be seen that $b\,n^2 = 2\,A_t\,m\,(d - n)$. Therefore

$$n^2 = \frac{2\,A_t\,m\,(d - n)}{b}$$

Now in this case A_t is also unknown, and is simply available as a proportion of the concrete, such proportion being expressed as r. We can therefore substitute this equivalent and obtain

$$n^2 = \frac{2\,r\,b\,d\,m\,(d - n)}{b}$$

Hence—

$$n^2 = 2\,r\,d^2\,m - 2\,r\,d\,m\,n$$
$$n^2 + 2\,r\,d\,m\,n = 2\,r\,d^2\,m$$

To both sides of the equation add $(r\,d\,m)^2$. Then

$$n^2 + 2\,r\,d\,m\,n + (r\,d\,m)^2 = (r\,d\,m)^2 + 2\,r\,d^2\,m.$$

The left-hand side of the equation can now be simplified as before, because $n + r\,d\,m$ is the square root of same, and therefore $(n + r\,d\,m)^2$ can be substituted as its value, giving $(n + r\,d\,m)^2 = (r\,d\,m)^2 + 2\,r\,d^2\,m$. Hence—

$$n + r\,d\,m = \sqrt{(r\,d\,m)^2 + 2\,r\,d^2\,m}$$
$$n = \sqrt{(r\,d\,m)^2 + 2\,r\,d^2\,m} - r\,d\,m$$
$$n = \left[\sqrt{(r\,m)^2 + 2\,r\,m} - r\,m\right] d$$

This, of course, is the same as the formula given by the London County Council, namely:

$$n = \left[\sqrt{m^2\,r^2 + 2\,m\,r} - m\,r\right] d$$

In the previous notes it was shown that when $r = \cdot 00675$, which is the economical proportion of steel to concrete, then n would always equal $\cdot 36\,d$. It will be advisable, therefore, to use the above formula and find what proportion n will be of d if the economical amount of steel is used as an illustration of the use of the formula, as follows:—

$$n = \left[\sqrt{m^2\,r^2 + 2\,m\,r} - m\,r\right] d$$

$m = 15$, and $r = \cdot 00675$, therefore

$$n = \left[\sqrt{15^2 \times \cdot 00675^2 + 2 \times 15 \times \cdot 00675 - 15 \times \cdot 00675}\right] d$$
$$n = \left[\sqrt{225 \times \cdot 0000455625 + \cdot 2025} - \cdot 10125\right] d$$
$$n = \left[\sqrt{\cdot 0102515625 + \cdot 2025} - \cdot 10125\right] d$$
$$n = \left[\sqrt{\cdot 2127515625} - \cdot 10125\right] d$$
$$n = \left[\cdot 46125 - \cdot 10125\right] d$$
$$n = \cdot 36\,d$$

A great number of figures are retained after the decimal point with the object of showing that the result absolutely agreed with that previously given. Formulæ, therefore, are available for finding the value of n in all cases, namely (1) when c and t only are known, (2) where the size of the beam and area of reinforcement are known and c and t are unknown, and (3) when c and t are unknown and the area of the steel is expressed as a proportion of the area of concrete.

Designing Beam for Certain Conditions.—As a final example, showing the application of the various formulæ, a case will be taken where it is required to design a beam to carry a uniformly distributed load of 350 lb. per foot run, in addition to its own weight, over a span of 24 ft. When designed the stresses in the steel and concrete will be checked to ascertain whether they are in accordance with the permissible limits.

It will here be seen that the weight of the beam itself has to be added to the external load before the bending moment can be found, and as the size is not yet known, an assumption must be made in order to do this. A little experience will enable the designer to gauge the size sufficiently near the actual size required to give a weight for the beam. In this instance the depth will be taken at 16 in., which is one eighteenth of the span, and b can be taken at 10 in., which is just over $\cdot 6\,d$. To the amount of d 2 in. should be added as a covering for the reinforcement, giving a total size of 18 in. by 10 in. The weight of the beam will be its cubical contents multiplied by 150 lb. per cub. ft. This equals 24 ft. × 1·5 ft. × ·63 ft. × 150 lb., or for 1-ft. run = 1·5 ft. × ·63 ft. × 150 lb. = 141·75 lb., say 142 lb.

The total load to be carried per foot run therefore equals 350 + 142 = 492 lb. This, multiplied by the span, will give the total load on the beam, thus 492 × 24 = 11,808 lb. The bending moment will equal $\frac{w\,l}{8}$.

$$\text{B M} = \frac{11808 \times 24 \times 12}{8}$$
$$\text{B M} = 425{,}088 \text{ in.-lb.}$$

The moment of resistance of the beam must equal 425,088 in.-lb., and the depth must first be calculated by the formula $d = \sqrt[3]{\frac{\text{B M}}{57}}$.

$$d = \sqrt[3]{\frac{425088}{57}}$$
$$d = \sqrt[3]{7457}$$
$$d = 19{\cdot}5 \text{ in.}$$

Take b at $\cdot 6\,d = 19{\cdot}5 \times \cdot 6 = 11{\cdot}7$ in. The area of steel will be given by taking the economical ratio, namely ·00675.

$$A_t = \cdot 00675\,b\,d$$
$$A_t = \cdot 00675 \times 11{\cdot}7 \times 19{\cdot}5$$
$$A_t = 1{\cdot}54 \text{ sq. in.}$$

If three bars, $\frac{13}{16}$ in. in diameter, are taken, this will give an area of 1·55 sq. in., which will be quite suitable. The beam, as now designed, has an effective depth of 19·5 in., with a breadth of 11·7 in. and steel reinforcement of 1·55 sq. in. Add 2 in. to the depth for covering the reinforcement, and make the breadth a practical size, and the beam becomes say 22 in. by 12 in.

Checking Beam after Designing.—It is now required to check the stresses in this beam, and as the actual size of the beam is now available it will be advisable to work out the bending moment again, because the calculated size is in excess of that assumed in the first instance.

Therefore the weight of the beam per foot run $= 1\frac{5}{6}$ ft. $\times$ 1 ft. $\times$ 150 lb. $=$ 275 lb. Total load per foot run $= 350 + 275 = 625$ lb. The total load to be carried equals the load per foot run multiplied by the span $=$ $625 \times 24 = 15{,}000$ lb. The bending moment $= \frac{w\,l}{8}$. Therefore $\text{B M} = \frac{15000 \times 24 \times 12}{8}$ $= 540{,}000$ in.-lb.

The position of the neutral axis must next be found by the formula

$$n = \sqrt{\frac{2\,A_t\,m\,d}{b} + \left(\frac{A_t\,m}{b}\right)^2} - \frac{A_t\,m}{b}$$

$$n = \sqrt{\frac{2 \times 1{\cdot}55 \times 15 \times 19{\cdot}5}{12} + \left(\frac{1{\cdot}55 \times 15}{12}\right)^2} - \frac{1{\cdot}55 \times 15}{12}$$

$$n = \sqrt{\frac{906{\cdot}75}{12} + \left(\frac{23{\cdot}25}{12}\right)^2} - \frac{23{\cdot}25}{12}$$

$$n = \sqrt{75{\cdot}562 + 3{\cdot}751} - 1{\cdot}93$$

$$n = \sqrt{79{\cdot}313} - 1{\cdot}93$$

$$n = 8{\cdot}9 - 1{\cdot}93$$

$$n = 6{\cdot}97 \text{ in.}$$

The stress in the concrete can now be found by the formula

$$c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right)}$$

$$c = \frac{2 \times 540000}{12 \times 6{\cdot}97 \times \left(19{\cdot}5 - \frac{6{\cdot}97}{3}\right)}$$

$$c = \frac{1080000}{12 \times 6{\cdot}97 \times 17{\cdot}18}$$

$$c = \frac{1080000}{1434{\cdot}87} = c = 753 \text{ lb.}$$

This is too high, as the permissible limit is 600 lb.

Now check the stress in the steel.

$$t = \frac{\text{B M}}{A_t \times \left(d - \frac{n}{3}\right)}$$

$$t = \frac{540000}{1{\cdot}55 \times \left(19{\cdot}5 - \frac{6{\cdot}96}{3}\right)}$$

$$t = \frac{540000}{1{\cdot}55 \times 17{\cdot}18}$$

$$t = \frac{540000}{26{\cdot}62} = t = 20{,}285 \text{ in.-lb.}$$

This, again, is much too high, as the permissible limit is 16,000 lb. for ordinary mild steel, and even for stronger steel the stress should not exceed 20,000 lb.

This example was given to show the value of checking the stresses after the beam has been designed, when the size has to be assumed, and consequently an error of judgment may take place, especially in the case of those inexperienced in design.

The next step is to see in what manner the beam can be adjusted to bring the stresses within the limits. The weight of the beam allowed in the first instance was insufficient, it being only 142 lb. per foot run, whereas the calculated beam was 275 lb. per foot run. Here, however, is some basis upon which to assume the size, and if the depth is increased to 22 in., and the breadth to $\cdot 6\,d$ = say 13·5 in., they cannot fail to be near the required size.

This may seem a great deal of work and calculation in connection with one beam, but it must be borne in mind that the example is merely given to illustrate principles and familiarise the reader with the formulæ employed.

Again, in a building where the plan is symmetrical, it usually happens that there are several beams which have the same span and carry the same amount of load, and consequently when the calculations for one beam are complete, many beams can be considered as designed, and it is worth some little trouble to produce economical and efficient design.

The reinforcement might be increased in this example, and thus the strength of the beam increased, but the economical percentage would not then be employed.

Of course, it is quite simple to design a member which is perfect and suitable for carrying a certain load as given in various textbooks, but a great number of these examples are done by selecting a beam with a certain ratio of reinforcement, finding the

moment of resistance, and working backwards, as it were, to find the safe load per foot run over a certain span. This method, of course, is not stated, but the examples frequently given in the textbooks are based on the particulars obtained, and thus no difficulty is encountered.

The object of the above example is rather to show how to deal with a case where the initial calculation is found to be incorrect, and to show in what manner these may be utilised in the second calculation. It will be seen that in the first instance the depth was based on the span, as the bending moment was unknown, and it goes to show how essential it is to have some connection between the depth and the bending moment if economical design is to result. However, the actual bending moment to be resisted is known when the beam had an effective depth of 19·5 in. and a breadth of 12 in., namely, 540,000 in.-lb. Calculate d according to this as a guide by the formula $d = \sqrt[3]{\frac{\text{B M}}{57}}$, then $d = \sqrt[3]{\frac{540000}{57}} = d = \sqrt[3]{9473}$, $d =$ 21 in. nearly. It must be realised, however, that the fact of increasing the size of the beam will again increase the weight of the beam, and therefore the bending moment, and an allowance must be made for this; and for this reason it will be advisable to take d at 22 in. as stated above, and b at $\cdot 6\, d$ = say 13·5 in.

The beam can now be worked out and again checked to see if the stresses are quite satisfactory. The size of the beam = 22 in. × 13·5 in., and the sectional area = 297 sq. in. A_t should equal ·00675 $b\,d$. This equals 297 × ·00675 = 2·00475 sq. in. The area of a round rod with a diameter of $\frac{15}{16}$ in. = ·6903 sq. in., and 3 rods = 2·0709 in., which is more than sufficient. Provision must be made for covering the bars with concrete, and if 2 in. be given, the total depth becomes 22 + 2 = 24 in. Having now designed the beam, the actual stresses can be checked, and to do this the actual bending moment must be found due to the given load to be carried, and the weight of the beam itself.

The latter per foot run = 2 × $1\frac{1}{8}$ × 150 lb. = say 337 lb. The total load per foot run = 350 + 337 = 687 lb. The total load = load per foot run × span = 687 lb. × 24 = 16,488 lb. The bending moment = $\frac{w\,l}{8}$, therefore B M = $\frac{16488 \times 24 \times 12}{8}$ = 593,568 in.-lb.

The position of the neutral axis must next be found by the formula

$$n = \sqrt{\frac{2\,A_t\,m\,d}{b} + \left(\frac{A_t\,m}{b}\right)^2} - \frac{A_t\,m}{b}$$

$$n = \sqrt{\frac{2 \times 2{\cdot}0709 \times 15 \times 22}{13{\cdot}5} + \left(\frac{2{\cdot}0709 \times 15}{13{\cdot}5}\right)^2} - \frac{2{\cdot}0709 \times 15}{13{\cdot}5}$$

$$n = \sqrt{\frac{1366{\cdot}794}{13{\cdot}5} + \left(\frac{31{\cdot}0635}{13{\cdot}5}\right)^2} - \frac{31{\cdot}0635}{13{\cdot}5}$$

$$n = \sqrt{101{\cdot}24 + 5{\cdot}294} - 2{\cdot}301$$

$$n = 10{\cdot}32 - 2{\cdot}301 = n = 8{\cdot}01 \text{ in.}$$

The stress in the concrete can now be found by the formula

$$c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right)}$$

$$c = \frac{2 \times 593568}{13{\cdot}5 \times 8{\cdot}01 \left(22 - \frac{8{\cdot}01}{3}\right)}$$

$$c = \frac{1187136}{13{\cdot}5 \times 8{\cdot}01 \times 19{\cdot}33}$$

$$c = \frac{1187136}{2090{\cdot}2}$$

$$c = 567{\cdot}9 \text{ lb.}$$

It will be advisable to check the stress in the steel as follows:

$$t = \frac{\text{B M}}{A^t \times \left(d - \frac{n}{3}\right)}$$

$$t = \frac{593568}{2{\cdot}0709 \times \left(22 - \frac{8{\cdot}01}{3}\right)}$$

$$t = \frac{593568}{2{\cdot}0709 \times 19{\cdot}33}$$

$$t = \frac{593568}{40{\cdot}03}$$

$$t = 14{,}828 \text{ lb.}$$

This amount is well below the permissible limit of 16,000 lb. The beam as designed will therefore have a total depth of 24 in., a breadth of 14 in., and be reinforced with three $\frac{15}{16}$-in. diameter rods.

Numerous problems and examples should be worked out with a view to becoming quite proficient in the use of the formulæ, and conversant with the method of making assumptions which are necessary in some cases to form a basis for the calculations, since, although the principles never vary, the method of procedure is likely to do so under different circumstances.

The beam would also be required to be calculated for shear and adhesion, but these will be dealt with subsequently. In any examples shown up to the present any stress other than that due to simple bending has been disregarded.

CALCULATIONS FOR SLABS

Slabs can be dealt with in a similar manner to beams as regards the calculations of strength, but the value of the bending moment will vary according to the proportion of the slab, if the slab is supported on all four sides; the formula for finding the depth in relation to the bending moment must be varied, as b will no longer be equal to $\cdot 6\,d$, but will always equal 12 in., as a strip of this width is calculated. In the case of a slab supported on all four sides and reinforced in two directions, the weight will be distributed on all four supporting edges, and consequently if a strip of the slab 12 in. wide is taken and the bending moment calculated, such bending moment can only be due to the reactions which occur at the ends of the strip. If the total weight upon the slab is distributed upon all four edges, it will follow that the reaction at any given point will be less than if the slab were supported on two sides only. A reduction in the bending moment must therefore be made to allow for this. If the slab is perfectly square, all four edges will carry an equal amount, and the reaction at any one side will be one-half the amount that would occur if the slab were supported on two sides only. If the reaction is only half the amount, then the bending moment will be reduced by a similar factor.

The bending moment at the centre of a beam or slab supported at the ends is equal to $\frac{w\,l}{8}$, and therefore the bending moment at the centre of a square slab, supported on all four edges, is equal to $\frac{w\,l}{16}$. Again, the bending moment at the centre of a beam or slab which is fixed at the ends is equal to $\frac{w\,l}{12}$, while with a square slab fixed on all four edges, the bending moment is equal to $\frac{w\,l}{24}$. These formulæ, of course, apply to cases when the load is uniformly distributed. This advantage may be considered as occurring when the length of the slab does not exceed twice the breadth; when the length exceeds this proportion, the slab is considered as one which is supported or fixed on two edges only. The advantages will vary between these two cases, namely, a square slab, and a slab where the length is twice the breadth, and the variation will depend upon the ratio of the length to breadth.

The draft regulations of the London County Council give formulæ showing the allowance that may be made as follows:—

w = weight on slab (total distributed weight, including its own weight).
b = breadth of slab.
l = length of slab.

Bending moment at any given cross section	*Condition of supports*	B.M. *for shorter span*	B.M. *for longer span*
At centre of span	Free	$\frac{w\,b}{8} \cdot \frac{1}{1+(b/l)^4}$	$\frac{w\,l}{8} \cdot \frac{1}{1+(l/b)^4}$
At end and centre of span	Partially fixed	$\frac{w\,b}{12} \cdot \frac{1}{1+(b/l)^4}$	$\frac{w\,l}{12} \cdot \frac{1}{1+(l/b^4)}$

Bending Moment at Centre of Slab.—It will be advisable to give an example to illustrate the use of these formulæ and show the decrease in the bending moment which is given by same. Assume a slab 8 ft. by 6 ft., which is supported on all four sides, and carries a uniformly distributed load of 250 lb. per sq. ft., and let it be required to calculate the bending moment at the centre of the span in both directions, namely, (1) by considering a portion 1 ft. wide and 6 ft. long, and (2) by considering a portion 1 ft. wide and 8 ft. long.

By referring to the table above, it will be seen that the formula for the shorter span is $\frac{w\,b}{8} \times \frac{1}{1+(b/l)^4}$. w = the total weight, and this will equal the area × by the weight per ft. super. Therefore w = 6 ft. × 1 ft. × 250 lb.; w = 1,500 lb.; b = the breadth

of the slab, which is 6 ft.; l = the length of the slab, which is 8 ft. Hence,

$$\frac{wb}{8} \times \frac{1}{1+(b/l)^4} = \frac{1500\text{ lb.} \times 72\text{ in.}}{8} \times \frac{1}{1+\left(\frac{6}{8}\right)^4}$$

$$= 13500 \times \frac{1}{1{\cdot}316} = 10{,}258 \text{ in.-lb.}$$

The allowance made in this instance is sufficient to reduce the bending moment from 13,500 in.-lb. to 10,258 in.-lb.

Now calculate the bending moment for the longer span, according to the formula $\frac{wl}{8} \times \frac{1}{1+(l/b)^4}$.

In this case w = 8 ft. × 1 ft. × 250 = 2,000 lb. Hence, $\text{B M} = \frac{2000 \times 96}{8} \times \frac{1}{1+\left(\frac{8}{6}\right)^4}$

$$\text{B M} = 24000 \times \frac{1}{1+3{\cdot}16} = 24000 \times \frac{1}{4{\cdot}16}$$

$$\text{B M} = 5{,}769 \text{ in.-lb.}$$

The reduction is considerably more in this instance, due, of course, to the fact that the edges of the short span will carry a greater portion of the load than the edges of the long span, and as the reactions will be less, so the bending moment will be reduced.

With regard to the thickness of the slab, this should be calculated according to the bending moment, and the following formula, with its construction, is given for this purpose. It has already been seen by previous deductions that $2\,\text{B M} = bcn\left(d - \frac{n}{3}\right)$, and that it is possible to substitute definite values for certain symbols as follows: b = 12 in., c = 600 lb., and $n = {\cdot}36\,d$, thus reducing the formula to

$$2\,\text{B M} = 12 \times 600 \times {\cdot}36\,d \times \left(d - \frac{{\cdot}36\,d}{3}\right)$$

$$2\,\text{B M} = 12 \times 600 \times {\cdot}36\,d \times {\cdot}88\,d$$

$$2\,\text{B M} = 7200 \times {\cdot}3168\,d^2$$

$$2\,\text{B M} = 2280{\cdot}96\,d^2 = \text{B M} = 1140{\cdot}48\,d^2$$

$$d = \sqrt{\frac{\text{B M}}{1140{\cdot}48}}$$

It will be quite near enough to use the formula as $d = \sqrt{\frac{\text{B M}}{1140}}$, the omission of the decimal figures making no appreciable difference in the result. In the case of a slab which is supported on all four edges, it has been shown that it is necessary to calculate the bending moment for both spans, and as the shorter span will give the greater bending moment, as shown by the example above, the depth should be calculated to suit this, and as this will be in excess of that required for the longer span, the longitudinal reinforcement required will be diminished accordingly. Again, it will obviously be an advantage, theoretically, to place the reinforcement across the slab below that parallel with the length of the slab, in order to obtain the maximum effective depth to resist the greater bending moment.

If the depth is found by calculation to be less than 3½ in., it should be increased to this amount for practical purposes, as any thickness under this is likely to be unreliable. The least diameter or thickness of the main reinforcement should not be less than ¼ in., and any other reinforcement should not be less than ⅛ in. in diameter. The maximum distance between the main tensile reinforcement should not be greater than 12 in., or less than 1 in., and the spacing of the reinforcement, when the load is uniformly distributed, may be gradually increased from the middle third to the outer edges, if the slabs of the bars are kept of a uniform size, provided that the spacing at the outer edges be not greater than three times the spacing at the centre of the slab.

Calculating Slab Supported on Four Sides.—As an example of slab design, let it be required to calculate the thickness and reinforcement for a slab 12 ft. by 8 ft., supported on all four sides to carry a load of 275 lb. per square foot in addition to its own weight. The stresses in the concrete and steel not to exceed 600 lb. and 16,000 lb. respectively.

The weight of the slab itself will require to be found in the first instance, as this must be added to the external load in order to ascertain the bending moment. Assume a total thickness of 6 in. for this purpose, and the weight per square foot will then be 1 ft. × 1 ft. × ·5 ft. × 150 lb. = 75 lb. The total load per square foot is equal to 275 lb. + 75 lb. = 350 lb. It is now required to calculate a portion of the slab, 8 ft. long and 1 ft. wide, and the area will equal 8 sq. ft. The total load on this portion will be the area multiplied by the weight per square foot; thus 8 × 350 = 2,800 lb. total load. The bending moment can now be calculated, bearing in mind that the slab is supported on all four sides, and the formula $\frac{wb}{8} \times \frac{1}{1+(b/l)^4}$ must be employed. There-

fore the bending moment will equal

$$\frac{2800 \times 96 \text{ in.}}{8} \times \frac{1}{1 + \left(\frac{8}{12}\right)^4}$$

$$\therefore \text{B M} = 33600 \times \frac{1}{1{\cdot}197}$$

$$\text{B M} = 28{,}070 \text{ lb.}$$

Having now obtained the bending moment, the depth can be calculated from it by the formula previously given, namely:

$$d = \sqrt{\frac{\text{B M}}{1140}}$$

$$d = \sqrt{24{\cdot}8}$$

$$d = 4{\cdot}9 \text{ in.}$$

To use the economical percentage of reinforcement, the sectional area must be made equal to ·00675 $b\,d$, and this, therefore, will be ·00675 × 12 × 4·9 = ·3969 sq. in. The area of one ½-in. round rod equals ·1963 sq. in., and two rods will give a sectional area of ·3926 sq. in. This is such a small fraction under the required amount that it can be used, and as d was found to be equal to 4·9 in., this may be increased to 5 in. for practical purposes. As there will be two ½-in. rods for every foot in width, they will be spaced at 6-in. centres, and this will be quite satisfactory. By adding 1 in. of concrete to the effective depth of 5 in., in order to afford covering for the reinforcement, the total depth will be equal to 6 in., which exactly agrees with the assumption made in the first instance.

It will now be necessary to check the stresses in the steel and concrete to see that they do not exceed the permissible limit.

The weight of the slab having been correctly assumed, it will not be necessary to recalculate the bending moment, as in the case of the beam designed in the previous example, and the position of the neutral axis is the only unknown factor. This will be found by using the formula already given :—

$$n = \sqrt{\frac{2\,\text{A}_t\,m\,d}{b} + \left(\frac{\text{A}_t\,m}{b}\right)^2} - \frac{\text{A}_t\,m}{b}$$

$$n = \sqrt{\frac{2 \times {\cdot}3926 \times 15 \times 5}{12} + \left(\frac{{\cdot}3926 \times 15}{12}\right)^2} - \frac{{\cdot}3926 \times 15}{12}$$

$$n = \sqrt{4{\cdot}9075 + ({\cdot}49075)^2} - {\cdot}49075$$

$$n = \sqrt{4{\cdot}9075 + {\cdot}2408} - {\cdot}49075$$

$$n = \sqrt{5{\cdot}1483} - {\cdot}49075$$

$$n = 2{\cdot}26 - {\cdot}49075$$

$$n = 1{\cdot}769$$

It is interesting to compare this value for n with that which would be given by taking same as ·36 d, such as would be the case if the stresses were known to be 600 and 16,000 lb. respectively; then it would equal ·36 × 5 in. = 1·8, which is very near.

To check the stress in the concrete will be the next step.

$$c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right)} =$$

$$c = \frac{2 \times 28070}{12 \times 1{\cdot}769 \times \left(5 - \frac{1{\cdot}769}{3}\right)}$$

$$c = \frac{56140}{12 \times 1{\cdot}769 \times 4{\cdot}4} = c = \frac{56140}{93{\cdot}4}$$

$$c = 601 \text{ lb.}$$

This can be considered as satisfactory, and the stress on the steel be checked by the formula:

$$t = \frac{\text{B M}}{\text{A}_t \times \left(d - \frac{n}{3}\right)}$$

$$t = \frac{28070}{{\cdot}3926 \times 4{\cdot}4} = t = \frac{28070}{1{\cdot}727}$$

$$t = 16{,}253 \text{ lb.}$$

This is slightly in excess of the permissible limit, and to overcome this the spacing of the bars may be slightly reduced, say from 6-in. to 5-in. centres. The student is advised to do this, and again check the value of t, as these examples are merely given for the purposes of illustration, and constant practice and experience will be the best asset of the designer when he has once mastered the principles and formulæ which govern his design.

It is necessary now to find the longitudinal reinforcement that will be necessary, and for this purpose the bending moment must be worked out for the longer span. As shown above, the total load per square foot = 350 lb., and the load on a strip 1 ft. wide and 12 ft. long will therefore equal 350 × 12 = 4,200 lb. The bending moment will be found by the formula for the longer span, which is $\frac{w\,l}{8} \times \frac{1}{1 + \left(\frac{l}{b}\right)^4}$.

$$\text{B M} = \frac{4{,}200 \times 144 \text{ in.}}{8} \times \frac{1}{1 + \left(\frac{12}{8}\right)^4}$$

$$\text{B M} = 75600 \times \frac{1}{1 + 5{\cdot}06}$$

$$\text{B M} = \frac{75600}{6{\cdot}06} = 12{,}475 \text{ in.-lb.}$$

It will not be necessary in this instance to calculate the depth, as this is given by the calculations for the shorter span, when it was found to be 5 in., and as the longitudinal reinforcement will be placed on the top of the cross-rods, d will be reduced by the distance between the centres of the two sets of rods, which will be $\frac{1}{2}$ in. if the longitudinal rods are of the same diameter. It will be better to keep them the same size if possible and space them farther apart. The depth will then be 5 in. − ·5 in., which equals 4·5 in. In order to make quite sure that this is sufficient, d may be calculated according to the bending moment given and the results compared. As before:

$$d = \sqrt{\frac{\text{B M}}{1140}}$$

$$d = \sqrt{\frac{12475}{1140}}$$

$$d = \sqrt{10} = 3{\cdot}1 \text{ in.}$$

This is less than the value already obtained, and the amount of 4·5 in. may be worked to, which will have the effect of reducing the amount of reinforcement required, as the lever arm of the internal resistances will be increased. This being so, seek for some method of deducing the value of A_t when d is given and is in excess of that required by calculating same from the bending moment. The bending moment is known, the depth is known, and n should be equal to ·36 d if the stresses are to be satisfactory, and the formula for finding t, which is $t = \dfrac{\text{B M}}{A_t \times \left(d - \frac{n}{3}\right)}$, can be used. If, however, a value is given to t of 16,000 lb., then there is only one unknown factor, namely A_t, and this can be found as follows:—

$$16000 = \frac{12475}{A_t \times \left(d - \frac{{\cdot}36\,d}{3}\right)}$$

$$A_t = \frac{12475}{16000 \times (d - {\cdot}12\,d)}$$

$$A_t = \frac{12475}{16000 \times {\cdot}88\,d}$$

$$A_t = \frac{12475}{16000 \times {\cdot}88 \times 4{\cdot}5} = A_t = \frac{12475}{63360}$$

$$A_t = {\cdot}1968 \text{ sq. in.}$$

The area of one $\frac{1}{2}$-in. diameter bar equals ·1963 sq. in., and therefore this size can be employed and spaced at 12-in. centres, which is the maximum permissible limit for tensional bars.

Checking the Slab.—It will now be necessary to check the stresses in the concrete and steel, and in order to do this the position of the neutral axis must first be found as follows:—

$$n = \sqrt{\frac{2\,A_t\,m\,d}{b} + \left(\frac{A_t\,m}{b}\right)^2} - \frac{A_t\,m}{b}$$

$$n = \sqrt{\frac{2 \times {\cdot}1963 \times 15 \times 4{\cdot}5}{12} + \left(\frac{{\cdot}1963 \times 15}{12}\right)^2} - \frac{{\cdot}1963 \times 15}{12}$$

$$n = \sqrt{2{\cdot}2083 + ({\cdot}2453)^2} - {\cdot}2453$$

$$n = \sqrt{2{\cdot}2684} - {\cdot}2453$$

$$n = 1{\cdot}5 - {\cdot}2453 = n = 1{\cdot}254 \text{ in.}$$

For checking the stress in the concrete:

$$c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right)}$$

$$c = \frac{2 \times 12475}{12 \times 1{\cdot}254 \times \left(4{\cdot}5 - \frac{1{\cdot}254}{3}\right)}$$

$$c = \frac{24950}{12 \times 1{\cdot}254 \times 4{\cdot}082} = c = \frac{24950}{61{\cdot}425}$$

$$c = 406 \text{ lb.}$$

This is well below the permissible limit, but t should be checked, and this will probably be very near the amount of 16,000 lb.

$$t = \frac{\text{B M}}{A_t \times \left(d - \frac{n}{3}\right)}$$

$$t = \frac{12475}{{\cdot}1963 \times \left(4{\cdot}5 - \frac{1{\cdot}254}{3}\right)}$$

$$t = \frac{12475}{{\cdot}1963 \times 4{\cdot}082}$$

$$t = \frac{12475}{{\cdot}8013}$$

$$t = 15{,}568 \text{ lb.}$$

This amount is not very much below the permissible value, and it will not be advisable, therefore, to decrease the reinforcement in any way, and the slab can be considered as satisfactorily designed. It will be impossible to stress the concrete to its full extent without overstressing the steel, as the depth, which influences the amount, of concrete is in excess of that theoretically required, as previously explained. Figs. 121 and 122 show how the dimensions are taken.

It is interesting to note what the result would be if the area of the reinforcement had been based on the theoretical depth instead of taking advantage of the depth given, and for the guidance of the student

this will be worked out. It has been seen that this depth was 3·1 in., and the theoretical area of the concrete in section therefore equalled 3·1 × 12 in. = 37·2 sq. in. If A_t was made equal to ·00675 $b\ d$, then it would be 37·2 × ·00675 = ·2511 sq. in. This is

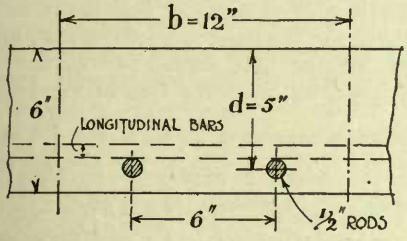


Fig. 121.—Section Through Cross Rods

greatly in excess of the amount actually required, and as the stresses as shown above are even now below the permissible limits, it is obvious that they would be far more so in this case, and the design would not be economical.

In the case of slabs which are fixed, the bending moment at the ends of the spans must also be calculated, and reinforcement provided on the upper surface to take the tension which will occur. This is effected by bending up some or all of the bars in the lower surface when they are no longer required in the latter position; or short bars may be placed on the upper surface and carried for the necessary distance into the slab. The turning up of the lower bars will assist in the resistance to shear, and this will be more fully explained and illus-

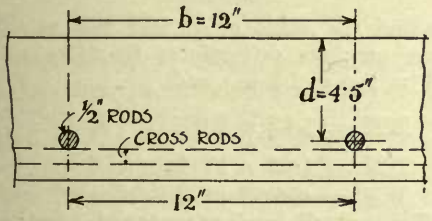


Fig. 122.—Section Through Longitudinal Rods

trated in the notes dealing with this portion of the subject. It must be borne in mind, however, that it will not be sufficient to calculate the bending moment in the centre of the span only, and although the same theory and principles will apply in both cases, the student is advised to take an example of a fixed slab and calculate the reinforcement necessary at the various points for the purposes of practice.

BEAMS WITH DOUBLE REINFORCEMENT

With regard to the beams and slabs described up to the present, no restrictions have been placed on the size of the members, and consequently the depth in some instances has been such as would cause great inconvenience in buildings of limited height. To allow of beams of less depth being employed, it is necessary to introduce double reinforcement, which consists of two sets of rods, one of these taking the tension as before, and the other taking a certain proportion of the compression which the reduced quantity of concrete is unable to resist. Another consideration is the weight of the beam. In the case of long spans, a beam with single reinforcement may be so heavy that its weight is greater than the external load to be carried; the introduction of compressional reinforcement reduces the size of the beam necessary and thus reduces the total load to be carried. This is an important consideration, as not only is the bending moment on the beam reduced, but so also is the load to be carried by the supports and the foundations.

The procedure in the design and calculations is somewhat different from that employed in the case of beams with single reinforcement, as with the latter it is necessary to begin with an assumption regarding the size of the beam required, in order to calculate the bending moment; but now a definite size can be fixed for the beam at the outset and suitable reinforcement inserted. This size should be made as near the economical section as the circumstances will permit, if there is any choice in the matter. As the size will thus be available at the outset, the weight of the beam can be found and the bending moment calculated. This bending moment will be the one to be resisted, and there will be no variation or re-calculation on this point.

The next step will be to consider the strength of the beam according to the fixed size by adding the economic proportion of einforcement, and considering it as a single reinforced beam, and then calculating the bending moment that such a beam will resist. It will necessarily be less than the actual bending moment to be resisted, and the difference between the two will give the

excess that has to be taken by the compressional reinforcement and the extra tensional bars that will be required over and above the economical percentage.

Figs. 123 and 124 will be some guide as to the disposition of the reinforcement, and the meaning of the symbols employed.

It is necessary to introduce a certain number of new formulæ, owing to the new factor in the compressional resistance, and these can be explained in the simplest manner by taking an example and working it out step by step with the necessary explanation as it becomes due.

Designing Beam with Double Reinforcement.—A beam is required to carry a uniformly distributed load of 10 tons in addition to its own weight over a span of 16 ft., while the over-all size is limited to 22 in. by 12 in. The actual weight of the beam must first be calculated in order to find the total load. This will equal $1\frac{5}{6} \times 1 \times 150$ lb. $\times$ 16 ft. $= 4{,}400$ lb., and the total load $= 10 \times 2240 + 4400 = 26{,}800$ lb. The bending moment will equal $\frac{wl}{8} = \text{B M} = \frac{26800 \times 16 \times 12}{8} = \text{B M} = 643{,}200$ in.-lb.

The actual bending moment to be resisted has thus been obtained, and it is now required to ascertain the moment of resistance of the specified beam if same is reinforced in the tensional area only, and see how far it is deficient. As the over-all depth is 22 in., an allowance must be made for covering the reinforcement, and this can be taken at 2 in., thus giving an effective depth of 20 in. and a breadth of 12 in.

The economical amount of reinforcement will be found by multiplying the effective sectional area of the concrete by ·00675, thus $20 \times 12 \times \cdot 00675 = 1\cdot 62$ sq. in. The moment of resistance of this beam will be equal to the sectional area of the steel multiplied by the safe stress per square inch multiplied by the lever arm of the internal resistance expressed as $A_t \times t \times \left(d - \frac{n}{3}\right)$, and the bending moment will equal the moment of resistance, therefore

$$\text{B M} = A_t \times t \times \left(d - \frac{n}{3}\right).$$

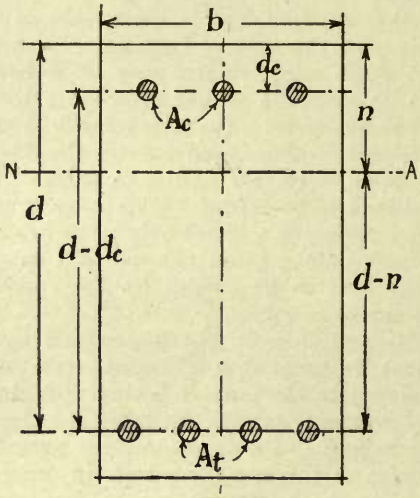


Fig. 123

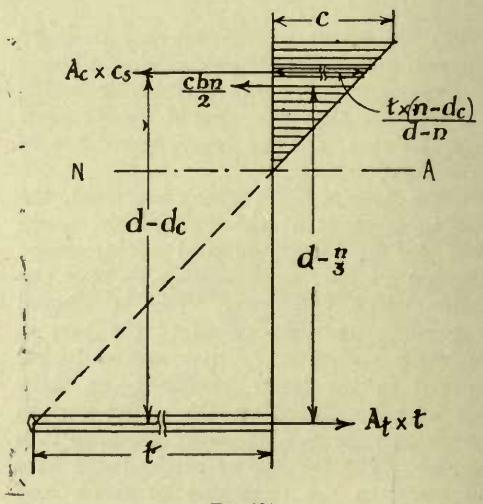


Fig. 124

Figs. 123 and 124.—Double Reinforcement in Beams

To find the value of n, take this as $\cdot 36\, d$ from previous deductions, therefore $n = \cdot 36 \times 20 = 7\cdot 2$ in., and

$$\text{B M} = 1\cdot 62 \times 16000 \times \left(20 - \frac{7\cdot 2}{3}\right)$$

$$\text{B M} = 1\cdot 62 \times 16000 \times 17\cdot 6$$

$$\text{B M} = 456192 \text{ in.-lb.}$$

The actual bending moment, however, equals 643,200, and the excess which has to be provided for by extra tensional reinforcement, and compressional reinforcement $= 643200 - 456192 = 187008$.

It may assist the student if the moment of resistance is worked out with regard to the concrete, in order to show that the same result is obtained and that the compressional reinforcement is necessary. Considering the concrete as taking 600 lb. per sq. in., this

gives $MR = \frac{c \times b \times n}{2} \times \left(d - \frac{n}{3}\right) = \frac{600 \times 12 \times 7{\cdot}2}{2} \times \left(20 - \frac{7{\cdot}2}{3}\right) = 25920 \times 17{\cdot}6 = 456192$ in.-lb. as before. It is therefore obvious that the compressional reinforcement will be required to take the excess of 187008 in.-lb.

The position of this compressional reinforcement must now be settled in order to determine the lever arm between the two sets of reinforcement. It is advisable to place this as far as possible from the neutral axis while obtaining a covering of concrete, and this will be given if it is situated at a distance of 2 in. from the upper surface, as

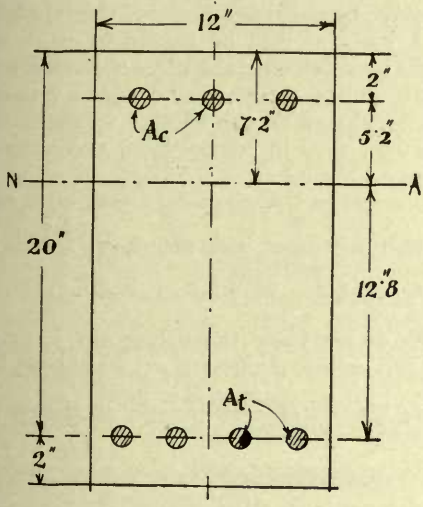


Fig. 125.—Designing Beam Having Double Reinforcement

shown in Fig. 125. There are thus two sets of reinforcement, one situated at a distance of 5·2 in. from the neutral axis and the other at a distance of 12·8 in. from the same point.

The distance from the upper surface to the centre of the compressional reinforcement is expressed by the symbol d_c, and the distance apart or lever arm of the two sets of bars will be $d - d_c$. Considering, first, the additional tensile reinforcement required, the limiting stress, which is 16,000 lb. per sq. in., is known, and also the lever arm, which is $d - d_c$. If, therefore, the excess bending moment is divided by $t \times (d - d_c)$, the amount of steel necessary will be found. This may be expressed as $A_t' = \frac{\text{Excess B M}}{t \times (d - d_c)}$. Working this out gives—

$$A_t' = \frac{187008}{16000 \times (20 - 2)}$$

$$A_t' = \frac{187008}{288000} = A_t' = {\cdot}649 \text{ sq. in.}$$

This additional reinforcement must be added to the amount already calculated for in the tensional area amounting to 1·62 sq. in. making a total of 2·269 sq. in.

The value of the compressional resistance must also equal the excess bending moment, and if the area of the steel in compression be expressed by the symbol A_c and the intensity of the stress by the symbol c_s, then $A_c \times c_s \times (d - d_c) =$ excess B M and $A_c = \frac{\text{Excess B M}}{c_s \times (d - d_c)}$. The value of c_s will require to be found before proceeding further, as although the stress intensity could safely be 16,000 lb. per sq. in. in compression as in tension, it will not be possible to develop this amount on account of the steel being so much nearer the neutral axis in the compression area than in the tension area. The stress varies directly as the distance from the neutral axis, and therefore, if the compression steel is placed twice as near to the axis as the tension steel, then it can only develop one-half the stress taken by the latter. In other words, the stress on the compression steel multiplied by the distance of the tension steel from the axis must equal the stress in the tension steel multiplied by the distance of the compression steel from the axis. Expressed by symbols, this reads $c_s \times (d - n) = t \times (n - d_c)$, therefore $c_s = \frac{t \times (n - d_c)}{d - n}$. In this example—

$$c_s = \frac{16000 \times (7{\cdot}2 - 2)}{20 - 7{\cdot}2}$$

$$c_s = \frac{832000}{12{\cdot}8}$$

$$c_s = 6{,}500 \text{ lb.}$$

Again, this may be shown in another way, as when steel and concrete are acting together in compression the stress in the steel can only reach m times that in the surrounding concrete, as it is only under these conditions that they can act together. The stress in the concrete at the extreme upper edge is equal to 600 lb., and consequently at a distance of d_c from the point it will be less, as it is closer to the neutral

axis. The stress will be equal to $\frac{c \times n - d_c}{n}$, which expresses proportion of stress at this point. The value of c_s as above will equal m times this amount, and therefore—

$$c_s = \frac{m \times c \times (n - d_c)}{n}$$

$$c_s = \frac{15 \times 600 \times (7{\cdot}2 - 2)}{7{\cdot}2}$$

$$c_s = 6{,}500 \text{ lb., as before.}$$

The formula $A_c = \frac{\text{Excess B M}}{c_s \times (d - d_c)}$ can now be worked out and the compressional steel found, thus:

$$A_c = \frac{187008}{6500 \times (20 - 2)}$$

$$A_c = \frac{187008}{117000}$$

$$A_c = 1{\cdot}598 \text{ sq. in.}$$

The beam as designed, therefore, has an over-all size of 22 in. by 12 in., with an effective depth of 20 in., and the reinforcement consists of 2·269 sq. in. in the tensional area and 1·598 sq. in. in the compressional area.

BEAMS AND SLABS WITH DOUBLE REINFORCEMENT

In checking the stresses in a beam already designed and containing double reinforcement, it is natural that the formulæ should be a little more complicated than in the case of single reinforced beams, as there are more factors to consider in the resistance. In the case of the stress on the concrete there is some portion of the compression taken by the steel in the upper part of the beam, and this, obviously, must be considered in arriving at the value of c. It will be necessary to see how the formula for finding c is constructed. As a basis for the reasoning, there is the theory already evolved, namely:

$\frac{c\,b\,n}{2} \times \left(d - \frac{n}{3}\right)$ and $A_c \times c_s \times (d - d_c)$ together equal B M. This can be stated as follows:—

$$2\,\text{B M} = c\,b\,n\left(d - \frac{n}{3}\right) + 2\,A_c \times c_s \times (d - d_c)$$

The factor c_s will be unknown until the compression stress on the steel is found, but its equivalent value, as previously shown, is $\frac{c \times m \times n - d_c}{n}$, and this can be substituted, giving:

$$2\,\text{B M} = c\,b\,n \times \left(d - \frac{n}{3}\right) + 2\,A_c\,c\,m \times \left(\frac{n - d_c}{n}\right) \times (d - d_c).$$

Therefore:

$$2\,\text{B M} = c\left[b\,n \times \left(d - \frac{n}{3}\right) + 2\,A_c\,m \times \left(\frac{n - d_c}{n}\right) \times (d - d_c)\right]$$

and c will be found by the formula:

$$c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right) + 2\,A_c\,m \times \left(\frac{n - d_c}{n}\right) \times (d - d_c)}$$

This formula appears to be a very large one, but it is perfectly simple, and will very quickly become familiar to the student, who, however, will be unable to make use of the equation until the value of n is known; and it will be necessary to deduce a formula for this, based on the relative values of the concrete, steel in compression, and steel in tension.

It is known that the total compression and tension are equal, and therefore $\frac{c\,b\,n}{2} + A^c \times c_s = t\,A_t$, or $c\,b\,n + 2\,A_c \times c_s = 2\,t\,A_t$.

We do not know the values of t, c, or c_s, but can replace them with equivalents as before; thus $\frac{t}{c} = \frac{m \times d - n}{n}$, as shown by previous reasoning, and $c_s = \frac{c\,m\,(n - d_c)}{n}$, or $\frac{c_s}{c} = \frac{m\,(n - d_c)}{n}$. Now $c\,b\,n + 2\,A_c \times c_s = 2\,t\,A_t$, and if we divide both sides of the equation by c we shall get $b\,n + \frac{2\,A_c \times c_s}{c} = \frac{2\,t\,A_t}{c}$, or $b\,n + 2\,A_c \times \frac{c_s}{c} = 2\,A_t \times \frac{t}{c}$.

By substituting the values of $\frac{t}{c}$ and $\frac{c_s}{c}$ as given above, we can remove the unknown factors and obtain the following:—

$$b\,n + \frac{2\,A_c\,m\,(n - d_c)}{n} = \frac{2\,A_t\,m\,(d - n)}{n}$$

Multiply both sides of the equation by n and we have

$$b\,n^2 + 2\,A_c\,m\,(n - d_c) = 2\,A_t\,m\,(d - n).$$

We have now got n on both sides of the equation, and this must be simplified as follows:—

$$b\,n^2 + 2\,A_c\,m\,n - 2\,A_c\,m\,d_c = 2\,A_t\,m\,d - 2\,A_t\,m\,n$$

$$b\,n^2 + 2\,A_c\,m\,n + 2\,A_t\,m\,n = 2\,A_t\,m\,d + 2\,A_c\,m\,d_c$$

$$b\,n^2 + 2\,m\,n\,(A_c + A_t) = 2\,m\,(A_t\,d + A_c\,d_c)$$

$$n^2 + \frac{2\,m\,n\,(A_c + A_t)}{b} = \frac{2\,m\,(A_t\,d + A_c\,d_c)}{b}$$

If we now add $\left[\frac{m\,(A_c + A_t)}{b}\right]^2$ to both sides of the equation it will be possible to take the square root of the left-hand side and further simplify as follows :—

$$n^2 + \frac{2\,m\,n\,(A_c + A_t)}{b} + \left[\frac{m\,(A_c + A_t)}{b}\right]^2 = \frac{2\,m\,(A_t\,d + A_c\,d_c)}{b} + \left[\frac{m\,(A_c + A_t)}{b}\right]^2$$

The square root of the left-hand side equals $n + \frac{m\,(A_c + A_t)}{b}$, therefore this will equal $\sqrt{\frac{2\,m\,(A_t\,d + A_c\,d_c)}{b} + \left[\frac{m\,(A_c + A_t)}{b}\right]^2}$, and

$$n = \sqrt{\frac{2\,m\,(A_t d + A_c d_c)}{b} + \left[\frac{m\,(A_c + A_t)}{b}\right]^2} - \frac{m\,(A_c + A_t)}{b}.$$

Having now obtained a formula for the position of the neutral axis, it will be possible to ascertain this and then calculate the stress on the concrete. When this has been done the stress on the steel can be found by considering the elasticity of the two materials and their relative distances from the neutral axis. Thus t will equal m times c multiplied by $\frac{d-n}{n}$, or, expressed as a formula, $t = c\,m\,\frac{d-n}{n}$, and c_s will equal $c\,m\,\frac{n-d_c}{n}$.

It will be advisable to give an example illustrating the use of these various formulæ for the purpose of checking; the case taken will be that of the beam previously designed to carry a load of 10 tons distributed in addition to its own weight over a span of 16 ft., when the effective depth was limited to 20 in. and the width to 12 in. The tensile reinforcement necessary was calculated to be 2·269 in., and the compressional reinforcement 1·598 in.

There will be no need to re-calculate the bending moment, as the size, and consequently the weight, of the beam was available at the outset. This bending moment was found to amount to 643,200 in.-lb. The first step will be that of finding the position of the neutral axis.

$$n = \sqrt{\frac{2\,m\,(A_t\,d + A_c\,d_c)}{b} + \left[\frac{m\,(A_c + A)}{b}\right]^2} - \frac{m\,(A_c + A_t)}{b}$$

$$n = \sqrt{\frac{2 \times 15\,(2{\cdot}269 \times 20 + 1{\cdot}598 \times 2)}{12} + \left[\frac{15\,(1{\cdot}598 + 2{\cdot}269)}{12}\right]^2} - \frac{15\,(1{\cdot}598 + 2{\cdot}269)}{12}$$

$$n = \sqrt{\frac{30 \times 48{\cdot}576}{12} + (4{\cdot}833)^2} - 4{\cdot}833$$

$$n = \sqrt{121{\cdot}44 + 23{\cdot}357} - 4{\cdot}833$$

$$n = \sqrt{144{\cdot}797} - 4{\cdot}833$$

$$n = 12{\cdot}03 - 4{\cdot}833$$

$$n = 7{\cdot}197 \text{ in.}$$

Upon reference to the figures taken when designing the beam, it will be seen that $n = 7{\cdot}2$ in., and this will afford some evidence to the student that the formula is satisfactory. The stress in the concrete can now be ascertained as follows :—

$$c = \frac{2\,B\,M}{b\,n \times \left(d - \frac{n}{3}\right) + 2\,A_c\,m \times \left(\frac{n - d_c}{n}\right) \times (d - d_c)}$$

$$c = \frac{2 \times 643200}{12 \times 7{\cdot}197 \times \left(20 - \frac{7{\cdot}197}{3}\right) + 2 \times 1{\cdot}598 \times 15 \times \left(\frac{7{\cdot}197 - 2}{7{\cdot}197}\right) \times (20 - 2}$$

$$c = \frac{1286400}{12 \times 7{\cdot}197 \times 17{\cdot}6 + 2 \times 1{\cdot}598 \times 15 \times {\cdot}71 \times 18}$$

$$c = \frac{1286400}{1520{\cdot}006 + 612{\cdot}673}$$

$$c = \frac{1286400}{2132{\cdot}679}$$

$$c = 603 \text{ lb.}$$

This is quite satisfactory, as it is only 3 lb. over the permissible limit of 600 lb. per square inch, and even this would not occur if all the decimal figures were retained in the calculations, but such a method would entail a great deal of unnecessary labour.

The stress in the tensional reinforcement $= c\,m\,\frac{d-n}{n}$; therefore

$$t = 603 \times 15 \times \frac{20 - 7{\cdot}197}{7{\cdot}197}$$

$$t = 603 \times 15 \times 1{\cdot}77$$

$$t = 16{,}000 \text{ lb.}$$

This can be considered satisfactory, and the stress in the compressional reinforcement will now be calculated by:

$$c_s = c\,m\,\frac{n - d_c}{n}$$

$$c_s = 603 \times 15 \times \frac{7{\cdot}197 - 2}{7{\cdot}197}$$

$$c_s = 603 \times 15 \times {\cdot}72$$

$$c_s = 6{,}512 \text{ lb.}$$

The beam then, as designed, can be considered as satisfactory. It will be noticed that the exact areas of the steel as calculated were retained and not substituted by actual practical size bars, as these would of necessity vary slightly, and this would affect the stresses. The object of this example has been to avoid all complications, as the theory is a little more difficult than with single reinforced beams, and the student should become quite clear on all the points in the formulæ and then work out various examples and check them, in some cases working out the bars required, and substituting these for the theoretical areas before finding the stresses in the concrete and steel.

All the theory and formulæ given will apply equally to slabs and beams, bearing in mind that the bending moments in the case of the former must, if necessary, be calculated in accordance with the rules previously given if supported or fixed on all four edges.

TEE BEAMS

These beams are those in which the compressional resistance is partly or wholly supplied by the slab above the beam, forming a large T-shaped section. Such a method of calculation is permissible and economical when the slab and beam are cast in one operation with no plane of cleavage between the two, and reinforcement is provided to resist the shearing stress occurring along the plane of junction.

When designing a reinforced concrete floor over a very large area it is necessary to divide the floor up into a number of panels by means of beams which will of necessity project down below the slabs which form the panels, and such beams will generally consist of main and secondary beams, the former spanning the greater distances and carrying the latter, which intersect at right angles.

In smaller floors the design may consist of one set of beams only, and in this case the slabs are considered to assist in the resistance of such beams, while in the case where two sets are employed it is usual to consider the secondary beams only as being assisted by the slabs, and it is essential that the slab reinforcement, which is at right angles to the beam, must extend through the full width of that portion of the slab which forms the compressional flange of the beam.

The width of slab that can be taken as acting with the beam will depend on circumstances, but it must not exceed either of the following : (1) One-third of the effective span of the tee beam, (2) three-fourths of the distance from centre to centre of the ribs of the tee beams, (3) fifteen times the thickness of the slab, (4) six times the width

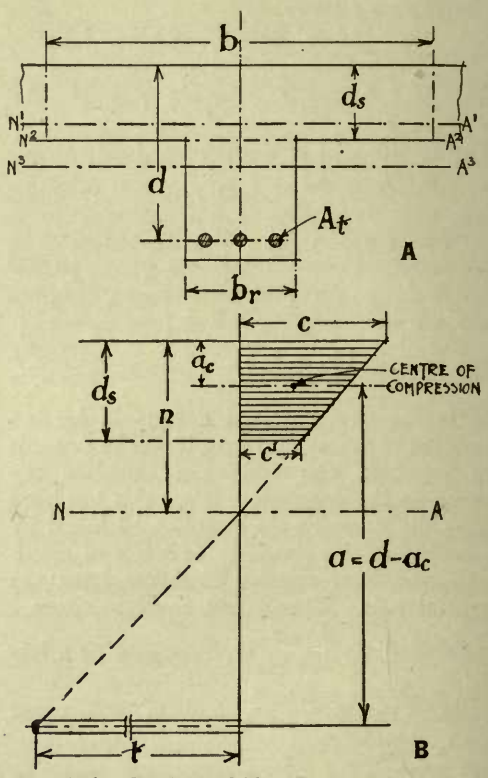


Fig. 126.—Section of Tee Beam and Method of Finding Total Compression

of the rib of the tee beam. It will be necessary, therefore, to determine which is the least of these and calculate accordingly.

The diagram presented by A, Fig. 126, shows the section of a tee beam, and will be helpful in explaining the value and meaning of the various symbols employed.

There are three cases that require to be considered according to the position of the neutral axis, and these are indicated on this diagram. The first is that when the neutral axis falls within the slab, the second where it coincides with the bottom of the slab, and the third where it falls below the slab.

The two former cases offer no difficulty, as all the formulæ that have previously been given for beams with single reinforcement can be employed, with the exception that b will now represent the width of that portion of the slab acting as the flange of the beam, and in the second case d_s, which expresses the thickness of the slab, will be equal to n.

In the third case, however, it will be seen that a portion of the rib will be acting in compression, and this will affect the centre of the compressional resistance, and consequently the lever arm of the internal forces.

In most cases the amount of concrete in the rib above the neutral axis which assists in compression is neglected in the actual calculations, as the area over which the stress is acting is comparatively small, and the resistance of such a small area situated so closely to the neutral axis is not such as to materially affect the economy of the design. The second report of the R.I.B.A. recommends that this method of calculation should be adopted, and it is therefore justifiable to consider the compression strength in this way.

Upon reference to B, Fig. 126, it will be seen that the total compression is represented by the shaded area, which has a depth equal to d_s and width equal to c at the extreme top edge and c^1 at the bottom edge, which coincides with the bottom of the slab. The area of the beam over which the stress is considered as acting is equal to $b \times d_s$, where b equals the width of the flange and d_s the thickness. Now, the total compression in the slab will be the area $b \times d_s$ multiplied by the stress per square inch. As before explained, the stress diminishes uniformly towards the neutral axis, and consequently the value of c^1 will be less than c in a direct ratio with its comparative distance from n. Therefore $c^1 = c \times \frac{n - d_s}{n}$, and the mean stress acting over the whole area will equal $\frac{c \times c^1}{2}$, or $\frac{c + c\frac{n - d_s}{n}}{2}$. If the area acting in compression be multiplied by the mean stress per square inch, then the total compression will be given. As before stated, the area $= b \times d_s$, therefore the total compression =

$$\frac{b \times d_s \times \left(c + c\,\frac{n - d_s}{n}\right)}{2}$$

The moment of the compression must equate with the bending moment, and the total compression will therefore be required to be multiplied by the lever arm of the internal forces. In the case of singly reinforced beams this was always equal to $d - \frac{n}{3}$, but this will no longer apply, as the centre of the compressional resistance will no longer be situated at a distance of $\frac{n}{3}$ from the upper surface. The symbol a, however, is used to express the value of the lever arm, and this can be employed to complete the formula for the moment of the compressional resistance, which will be equal to

$$\frac{b \times d_s \times \left(c + c\,\frac{n - d_s}{n}\right)}{2} \times a,$$ and this must equal B M.

From this formula may be deduced a formula for finding the value of c when the bending moment has been calculated, as follows :—

$$\frac{c\left[b \times d_s \times \left(2\,\frac{n - d_s}{n}\right) \times a\right]}{2} = \text{B M}$$

$$\frac{c\left[b \times d_s \times (2\,n - d_s) \times a\right]}{2\,n} = \text{B M}$$

Therefore $c = \dfrac{2\,\text{B M}\,n}{b \times d_s \times (2\,n - d_s) \times a}$

The value of the total compression has now been obtained, neglecting the concrete in the rib, and also the method of finding the value of c when the beam is designed. It is now necessary to show how the lever arm of the internal forces can be ascertained, as it will not be $d - \frac{n}{3}$, as in previous cases, owing to the total compression, as shown at B, Fig. 126, no longer being represented by a triangle. As before, the whole of the compression must be considered as acting at the centre of gravity, and it is necessary, therefore, to find the centre of gravity of the shaded portion, shown on the diagram. The draft regulations of the London County Council state that the lever arm can be taken approximately as $d - \frac{d}{2}$. This is a useful rule for preliminary calculations when the steel is required to be found and the position of the neutral axis is not available, but an accurate method must be deduced for exact calculations. Now if

the shaded portion was a triangle having a base equal to c and a height equal to d_s, then the centre of gravity would be situated at a distance of $\frac{d_s}{3}$ from the top edge. It is necessary to consider, however, a figure having four sides, where c^1 will bear a certain relation to c, and it is obvious that the centre will be nearer to the top than $\frac{d_s}{3}$, and by the principle of moments it will be found that the actual position of the point from the top edge will equal $\frac{d_s}{3} \times \frac{c + 2c^1}{c + c^1}$. Express the distance from the top edge by the symbol a_c, as shown in Fig. 126. It has previously been seen, however, that $c^1 = c\frac{n - d_s}{n}$, and by substitution the following is obtained :—

$$a_c = \frac{d_s}{3} \times \frac{c + 2c \times \frac{n - d_s}{n}}{c + c \times \frac{n - d_s}{n}} = \frac{d_s}{3} \times \frac{3cn - \frac{2d_s}{n}}{2cn - \frac{d_s}{n}} = \frac{d_s}{3} \times \frac{3n - 2d_s}{2n - d_s} = \frac{3d_s n - 2d_s^2}{6n - 3d_s}$$

Therefore $a_c = \frac{3d_s n - 2d_s^2}{6n - 3d_s}$, and the lever arm of the internal forces will equal $d - a_c$ or $d - \left(\frac{3d_s n - 2d_s^2}{6n - 3d_s}\right)$.

The total tension will equal $t \times A_t$, and the tensional resistance will be this value multiplied by the lever arm, namely $t \times A_t \times a$, where $a = d - a_c$. This tensional resistance must also equal the bending moment, and therefore $t\,A \times d - a_c = B\,M$, and the stress in the steel can be found in a beam that has already been designed by the formula

$$t = \frac{B\,M}{A_t \times d - a_c}.$$

Having ascertained the methods of dealing with both the compressional and tensional resistances, the next step will be to deduce some process for finding the position of n in the case of a beam already designed, and this is of great importance, as there are three possible cases, as stated previously. In order to be quite accurate, it will be necessary to consider the concrete in the rib, as it is obvious that any compression in this portion would affect the position of n, and although it is neglected in the actual calculations for strength, it must be taken into account in the neutral axis formula. The compression in the slab has been shown to be equal to

$$\frac{b\,d_s\left(c + c\,\frac{n - d_s}{n}\right)}{2}$$

and to this must be added the compression on the leg. This will amount to

$$\frac{b_r \times c \times \frac{n - d_s}{n}}{2}$$

where b_r is equal to the width of the rib, giving a total compression of

$$\frac{b\,d_s\left(c + c\frac{n - d_s}{n}\right) + b_r\,c\left(\frac{n - d_s}{n}\right)}{2}$$

and as the total compression and tension are equal, thus

$$\frac{b\,d_s\left(c + c\frac{n - d_s}{n}\right) + b_r\,c\left(\frac{n - d_s}{n}\right)}{2} = t\,A_t.$$

This can be simplified and deduced to give a value for n as follows :—

$$\frac{2cb\,d_s n - cb\,d_s^2 + cb_r n - cb_r d_s}{2n} = t\,A_t$$

$$\frac{c\,(2b\,d_s n - b\,d_s^2 + b_r n - b_r d_s)}{2n} = t\,A_t$$

$$2b\,d_s n - b\,d_s^2 + b_r n - b_r d_s = \frac{2t\,A_t n}{c}$$

but $\frac{t}{c} = \frac{m\,(d - n)}{n}$, and by substitution we get :

$$2b\,d_s n - b\,d_s^2 + b_r n - b_r d_s = \frac{2m\,A_t n\,(d - n)}{n}$$

$$2b\,d_s n - b\,d_s^2 + b_r n - b_r d_s = \frac{2m\,A_t n\,d - 2m\,A_t n^2}{n}$$

$$2b\,d_s n - b\,d_s^2 + b_r n - b_r d_s = 2m\,A_t d - 2m\,A_t n$$

$$2b\,d_s n + b_r n + 2m\,A_t n = b\,d_s^2 + b_r d_s + 2m\,A_t d$$

$$n\,[2\,(b\,d_s + m\,A_t) + b_r] = d_s\,(b\,d_s + b_r) + 2m\,A_t d$$

$$\text{therefore } n = \frac{d_s\,(b\,d_s + b_r) + 2m\,A_t d}{2\,(b\,d_s + m\,A_t) + b_r}$$

Thus all the formulæ necessary is obtained for the checking of beams which are already designed, and it is only necessary to ascertain the method to be followed when designing the beams to carry a stated load, and this will now be given.

In the designing of tee beams, there are certain factors which will be available from the previous calculations, and it will be advisable to note these in the first instance. The width of the compression flange, together with the thickness, will be given, as the slabs will have been designed previously, and, consequently, the values for b and d_s will be known. There remains for consideration the method of finding the thickness of the rib; the value of d required to give economical design; and the amount of reinforcement. The first-mentioned, namely, the width of the rib, will generally be fixed by the designer according to experience, as it will be influenced by the fact that the rods in the tensional area must be properly spaced and covered; and as the value of the compression in the rib is neglected—even when the neutral axis falls below the slab—the calculations for the design will not be affected. There is a minimum limit, however, and that is, the width must not be less than one-sixth of the width of the compression flange, but it is seldom that this minimum will be employed.

The method of finding the economical depth is one that calls for a little more consideration. In the first instance, it is not known whether the neutral axis will fall below the slab or not, and if it should be found to do so after the design is complete, then the lever arm will not be $d - \frac{n}{3}$, but $d - a_c$, and, unfortunately, no definite relation between a_c and d can be fixed unless the amount of reinforcement, and consequently the position of the neutral axis, is known. To keep matters as simple as possible, however, it will be advisable still to consider n as equal to $\cdot 36\, d$, as will be the case if it falls within the slab, and to construct a formula for finding d accordingly; and if the value for d thus found is not quite accurate, it will form the best guide, and the necessary adjustment can be made after.

In previous notes it has been shown that $\text{B M} = \frac{c\,b\,n}{2} \times \left(d - \frac{n}{3}\right)$ for a single reinforced beam, and if the neutral axis is assumed to fall within the depth of the slab, then this formula will hold good. In deducing a formula for giving the value of d, values may be substituted for c and n, but b will be unknown, as it will vary in practically every example, and will not necessarily bear any definite relation to d. By putting in the values already known, it can be deduced as follows:

$$2\,\text{B M} = c \times b \times n \times \left(d - \frac{n}{3}\right)$$

$c = 600$ and $n = \cdot 36\, d$, then

$$2\,\text{B M} = 600 \times b \times \cdot 36\, d \times \left(d - \frac{\cdot 36\, d}{3}\right) =$$

$$2\,\text{B M} = 600 \times b \times \cdot 36\, d \times \cdot 88\, d$$

$$2\,\text{B M} = 180{\cdot}08\, b\, d^2$$

$$\text{B M} = 90{\cdot}04\, b\, d^2$$

$$d^2 = \frac{\text{B M}}{90{\cdot}04\, b} = d = \sqrt{\frac{\text{B M}}{90{\cdot}04\, b}}.$$

It will be sufficiently accurate if the decimal figures are deleted, giving

$$d = \sqrt{\frac{\text{B M}}{90\, b}}$$

With regard to the method of calculating the amount of reinforcement required, or the value of A_t, this again presents difficulties, as it will depend upon certain factors, one of which will be unavailable, namely, the lever arm; and although it may be taken as being equal to $\cdot 00675\, b\, d$ when the neutral axis falls within the slab, this method will not be absolutely accurate when the axis falls below the slab. Another method will be that of assuming the value of the lever arm, and for the preliminary calculations this may be taken as equal to $d - \frac{d_s}{2}$ when a safe approximation will result under ordinary circumstances. If this method is considered and accepted, then A can be found as follows:

$\text{B M} = A_t \times t \times a$, where a = the lever arm, and this is assumed as equal to $d - \frac{d_s}{2}$,

therefore $\text{B M} = A_t \times t \times \left(d - \frac{d_s}{2}\right)$,

then $A_t = \dfrac{\text{B M}}{t \times \left(d - \frac{d_s}{2}\right)}$.

TEE BEAMS CONTINUOUS OR FIXED AT ENDS

In the case of tee beams that are continuous or fixed at the ends, it will be necessary to calculate the strength at the supports; and as the flange at these points will be in the tensional area, its value will be lost and the calculations must be made for a rectangular beam having a width equal to the width of the rib only. If an example is now

taken, and the beam is designed and checked according to the theory and formulæ given, the student will be able to realise the application quite clearly.

It is required to design a tee beam, fixed at the ends, to carry a uniformly distributed load of 2,000 lb. per ft. run, including the weight of the beam itself, over an effective span of 20 ft. The slab has been designed, and has a total depth of 6 in., while the width acting in compression with the beam can be taken at ten times d_s, which equals 60 in.

The total load will equal the load per foot run multiplied by the effective span, thus $w = 2000 \times 20 = 40000$ lb. As the beam is fixed at the ends, the bending moment at the centre can be considered as equal to $\frac{wl}{12}$, then $\text{B M} = \frac{40{,}000 \times 20 \times 12}{12} =$ 800000 in.-lb.

The first step in the actual design of the beam will be to calculate the depth by the formula $d = \sqrt{\frac{\text{B M}}{90\,b}}$, then $d = \sqrt{\frac{800000}{90 \times 60}}$ $= d = \sqrt{148} =$ say 12 in.

Let us next find the value of A_t by considering same as $\cdot 00675\, b\, d$. Then $A_t = \cdot 00675 \times 60 \times 12 = A_t = 4{\cdot}86$ in. If the value of the steel required is also calculated by the alternative approximate method of considering the lever arm as equal to $d - \frac{d_s}{2}$, then the use of the formula will be shown.

This formula was $A_t = \frac{\text{B M}}{t \times \left(d - \frac{d_s}{2}\right)}$

$$\text{Therefore } A_t = \frac{800000}{16000 \times (12 - \frac{6}{2})}$$

$$A_t = \frac{800000}{16000 \times 9}$$

$$A_t = 5{\cdot}55 \text{ in.}$$

The first method, which gave 4·86 in., is therefore the most economical, provided the permissible stresses are not exceeded, and this will be ascertained by checking the beam. Should the steel provided by the first method be found insufficient, then it may be increased by any amount up to that given by the second method, when it will always be found to be sufficient.

In checking the beam the position of the neutral axis must first be found, and the formula for this has been given, namely:

$$n = \frac{d_s\,(b\,d_s + b_\gamma) + 2\,m\,A_t\,d}{2\,(b\,d_s + m\,A_t) + b_\gamma}.$$

Assuming b_γ to be 10 in., then

$$n = \frac{6\,(60 \times 6 + 10) + 2 \times 15 \times 4{\cdot}86 \times 12}{2\,(60 \times 6 + 15 \times 4{\cdot}86) + 10}$$

$$n = \frac{6 \times 370 + 30 \times 58{\cdot}32}{2 \times (360 + 72{\cdot}9) + 10}$$

$$n = \frac{2220 + 1749{\cdot}6}{865{\cdot}8 + 10} = n = \frac{3969{\cdot}6}{865{\cdot}8} = n = 4{\cdot}58 \text{ in.}$$

The thickness of the slab was given as 6 in., and the distance from the top edge to the neutral axis is 4·58 in., therefore the latter falls within the slab, and the beam can be considered as a singly reinforced beam when the lever arm equals $d - \frac{n}{3}$. The stresses can be checked as follows:

$$c = \frac{2\,\text{B M}}{b\,n \times \left(d - \frac{n}{3}\right)}; \text{ therefore}$$

$$c = \frac{2 \times 800000}{60 \times 4{\cdot}58 \times \left(12 - \frac{4{\cdot}58}{3}\right)}$$

$$c = \frac{1600000}{60 \times 4{\cdot}58 \times 10{\cdot}48}$$

$$c = \frac{1600000}{2879{\cdot}9}$$

$$c = 555 \text{ lb.}$$

This is quite satisfactory, and t can now be calculated.

$$t = \frac{\text{B M}}{A_t \times \left(d - \frac{n}{3}\right)}; \text{ therefore}$$

$$t = \frac{800000}{4{\cdot}86 \times \left(12 - \frac{4{\cdot}58}{3}\right)}$$

$$t = \frac{800000}{4{\cdot}86 \times 10{\cdot}48}$$

$$t = \frac{800000}{50{\cdot}835}$$

$t = 15737$ lb., which is also quite satisfactory.

The steel required was 4·86 in., and the area of one bar having a diameter of $1\frac{1}{8}$ in. $= \cdot 994$ in., and five bars of this size would give an area $= \cdot 994 \times 5 = 4{\cdot}97$ in., which will do very well. As five rods are to be used, it will be necessary to put them in two rows one over the other, and the depth of 12 in. will be the distance from the top edge to the centre between these rows. In order to get sufficient covering for the lower rods, it will be necessary to add 3 in. of concrete, giving a total depth of 15 in.

In this example the beam would require to be calculated at the ends, as it was stated to be fixed, and this would be done by using the formulæ given for doubly reinforced beams. As this example, however, gave an instance where the neutral axis fell within the slab, it will possibly be more useful to give another example where the axis falls below the slab and calculate the section at the centre and the ends also, and the student will then have an illustration of the procedure in each case. It must be borne in mind that these examples are taken at random, and, as before stated, are given for the sole purpose of showing the application of the formulæ and the methods adopted, and the reader will be well advised if he studies the working drawings of reinforced concrete work, and endeavours to follow the practical application of the theory. It is somewhat difficult to select an example where the axis will fall below the slab unless the load to be carried is made exceptionally heavy, or the compression flange is taken as being very thin and narrow; and if the load is excessively heavy and the beam is fixed at the ends, then a large amount of reinforcement will be required in the compressional area at the ends where the flange is situated in the tensional area, as the steel will not be situated at any great distance from the neutral axis and cannot be stressed to its full limit.

As an example, however, let it be required to design a tee beam securely fixed at the ends, to carry a uniformly distributed load of 2,551 lb. per ft. run, including the weight of the beam itself over an effective span of 14 ft. The section at the centre of the span is to be calculated in the first instance, and then the section at the ends, and the slab can be taken as 4 in. thick, and the width, acting with the beam, as ten times d_s, which equals 40 in.

The total load will equal the load per ft. run multiplied by the span = 2551 lb. × 14 ft. = 35714 lb.

The bending moment at the centre will equal $\frac{wl}{12}$, therefore $\text{B M} = \frac{35714 \times 14 \times 12}{12}$ = 499,996 in.-lb., say 500,000 in.-lb.

First calculate the depth, namely:

$$d = \sqrt{\frac{\text{B M}}{90\,b}} \text{ therefore } d = \sqrt{\frac{500000}{90 \times 40}}$$

$$d = \sqrt{139} = 11{\cdot}8 \text{ in., say 12 in.}$$

The value of A_t must next be found by taking same at ·00675 $b\,d$; therefore A_t = ·00675 × 40 × 12 = 3·24 in.

The section at the centre is now obtained, and before proceeding with the calculations at the ends of the span it will be advisable to find the position of the neutral axis, and see that this falls below the slab, and also check the stresses on the steel and concrete.

The position of n will be found by the formula $n = \frac{d_s\,(b\,d_s + b_r) + 2\,m\,A_t\,d}{2\,(b\,d_s + m\,A_t) + b_r}$

There is only one unknown factor that must be settled before working out the formula, and that is the value of b_r. Taking this, as before, at 10 in., then

$$n = \frac{4\,(40 \times 4 + 10) + 2 \times 15 \times 3{\cdot}24 \times 12}{2\,(40 \times 4 + 15 \times 3{\cdot}24) + 10}$$

$$n = \frac{4 \times 170 + 30 \times 38{\cdot}88}{2\,(160 + 48{\cdot}6) + 10}$$

$$n = \frac{680 + 1166{\cdot}4}{427{\cdot}2}$$

$$n = \frac{1846{\cdot}4}{427{\cdot}2}$$

$$n = 4{\cdot}32 \text{ in.}$$

As the thickness of the slab is only 4 in., it will be seen that the axis falls ·32 in. below same, and although this is a very small distance, it will serve as an example.

The position of the neutral axis having been obtained, the next thing will be to check the stress in the concrete, and as the axis falls below the slab it will be necessary to use the formula

$$c = \frac{2\,\text{B M}\,n}{b \times d_s \times (2\,n - d_s) \times a}$$

The factor a is at present unknown, as this represents the lever arm of the internal forces, and this must be found in the first instance. It is known that $a = d - a_c$ and $a_c = \frac{3\,d_s\,n - 2\,d_s^2}{6\,n - 3\,d_s}$, therefore

$$a_c = \frac{3 \times 4 \times 4{\cdot}32 - 2 \times 4 \times 4}{6 \times 4{\cdot}32 - 3 \times 4}$$

$$a_c = \frac{51{\cdot}84 - 32}{25{\cdot}92 - 12} = a_c = \frac{19{\cdot}84}{13{\cdot}92}$$

$$a_c = 1{\cdot}42 \text{ in.}$$

As a comparison, it is interesting to note that if the neutral axis had coincided with the under side of the slab, the centre of compression would have been equal to $\frac{d_s}{3}$ which would give a distance of 1·44 in. from the top edge; and as the axis falls below the slab by a distance of ·32 in., the centre of

compression is ·02 in. nearer the top edge, and consequently the lever arm is increased by this amount. As already stated, $a = d - a_c$, therefore a = 12 in. − 1·42 in. = 10·58 in., and this value being now available, c can be found as follows:

$$c = \frac{2 \times 500000 \times 4{\cdot}32}{40 \times 4 \times (2 \times 4{\cdot}32 - 4) \times 10{\cdot}58}$$

$$c = \frac{4320000}{160 \times 4{\cdot}64 \times 10{\cdot}58}$$

$$c = \frac{4320000}{7854{\cdot}592}$$

c = 549 lbs.

This is well below the permissible limit,

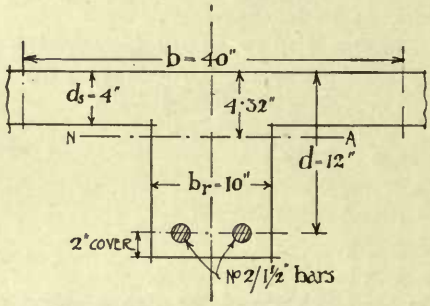


Fig. 127.—Section, at Centre of Span, of Tee Beam Fixed at Ends

and the value of t can be next calculated by the formula $t = \frac{\text{B M}}{A_t \times a}$ when a again $= d - a_c$, which = 10·58 in., therefore $= \frac{500000}{3{\cdot}24 \times 10{\cdot}58} = t = \frac{500000}{34{\cdot}2792}$, t = 14586 lb. This also is well below the permissible limit, and the section at the centre may be considered as quite satisfactory. If two bars having a diameter of 1½ in. are used, then the sectional area of steel will equal 1·7671 × 2 = 3·53 sq. in., which is more than sufficient.

The section as designed is illustrated in Fig. 127, and the calculations for the section at the ends of the span may now be considered. In this latter case, a different bending moment will exist having a value equal to $\frac{wl}{10}$, therefore B M $= \frac{35714 \times 14 \times 12}{10}$ = 599995 in.-lb., say 600000 in.-lb. This is a greater bending moment than that required to be resisted in the centre of the span, and as the tension will now be in the upper surface, a comparatively large amount of reinforcement will be required. The section will be calculated as a beam with double reinforcement where the size of the concrete is fixed. The portions of the slab on each side of the beam must be neglected, leaving a section 14 in. deep and 10 in. wide. Allowing 2 in. from the top edge to the centre of the reinforcement, the effective section becomes 12 in. by 10 in. = 120 sq. in. The economical percentage if singly reinforced would be ·00675 × 120 = ·81 in. The bending moment that such a beam would resist will equal $A_t \times t \times \left(d - \frac{n}{3}\right)$ and n would = ·36 d = 12 × ·36 = 4·32 in., then B M = ·81 × 16000 × $\left(12 - \frac{4{\cdot}32}{3}\right)$ = B M = ·81 × 16000 × 10·56 = 136857 in.-lb. The actual bending moment to be resisted, however, is 600000, and the excess which will have to be taken by the compressional steel and additional tensional steel equals 600000 − 136857 = 463143 in.-lb. If the compressional steel is placed 2 in. from the bottom edge, then d_c will equal 2 in., and the lever arm or distance between the two sets of reinforcement will equal $d - d_c$ = 12 − 2 = 10 in. The additional area of steel required in tension will be found by the formula:

$$A_t^1 = \frac{\text{Excess B M}}{t \times (d - d_c)} = A_t^1 = \frac{463143}{16000 \times (12 - 2)} =$$

$$A_t^1 = \frac{463143}{160000} = A_t^1 = 2{\cdot}89 \text{ sq. in.}$$

This amount, plus ·81 in., the area found above, will give a total amount of 3·7 in. for the steel required in the upper or tensional area. The steel required in the compressional area will be found by the formula:

$$A_c = \frac{\text{Excess B M}}{c_s \times (d - d_c)}$$

The value of c_s must be found, and, as before stated, this will depend on the position of A_c in relation to the neutral axis, namely,

$$c_s = \frac{t \times (n - d_c)}{d - n}; \text{ therefore}$$

$$c_s = \frac{16000 \times (4{\cdot}32 - 2)}{12 - 4{\cdot}32} = c_s = \frac{37120}{7{\cdot}68}$$

c_s = 4833 lb.

It will be seen that the steel will be stressed at a very low figure. The formula for the amount of steel can now be worked out as follows:

$$A_c = \frac{463143}{4833 \times (12 - 2)}$$

$$A_c = \frac{463143}{48330} = A_c = 9{\cdot}58 \text{ sq. in.}$$

Before actually giving the finished section, it will be advisable to check the stresses, and see that they do not exceed the permissible limits, and in order to do this, the position of the neutral axis must first be calculated by the formula given for double reinforced beams, namely :

$$n = \sqrt{\frac{2\,m\,(A_t\,d + A_c\,d_c)}{b} + \left[\frac{m\,(A_c + A_t)}{b}\right]^2} - \frac{m\,(A_c + A_t)}{b}\text{; therefore}$$

$$n = \sqrt{\frac{2 \times 15\,(3{\cdot}7 \times 12 + 9{\cdot}58 \times 2)}{10} + \left[\frac{15\,(9{\cdot}58 + 3{\cdot}7)}{10}\right]^2} - \frac{15\,(9{\cdot}58 + 3{\cdot}7)}{10}$$

$$n = \sqrt{\frac{30\,(44{\cdot}4 + 19{\cdot}16)}{10} + \left[\frac{15 \times 13{\cdot}28}{10}\right]^2} - \frac{15 \times 13{\cdot}28}{10}$$

$$n = \sqrt{\frac{1906{\cdot}8}{10} + \left[\frac{199{\cdot}2}{10}\right]^2} - \frac{199{\cdot}2}{10}$$

$$n = \sqrt{190{\cdot}68 + 396{\cdot}7064} - 19{\cdot}92$$

$$n = \sqrt{587{\cdot}38} - 19{\cdot}92 = n = 24{\cdot}23 - 19{\cdot}92$$

$$n = 4{\cdot}31 \text{ in.}$$

This compares very well with the position calculated according to the economical percentage when designing, namely 4·32 in. The stress in the concrete can now be found as follows :

$$c = \frac{2\,B\,M}{b\,n \times \left(d - \frac{n}{3}\right) + 2\,A_c\,m\left(\frac{n - d_c}{n}\right) \times d - d_c}$$

$$c = \frac{2 \times 600000}{10 \times 4{\cdot}31 \times \left(12 - \frac{4{\cdot}31}{3}\right) + 2 \times 9{\cdot}58 \times 15 \times \left(\frac{4{\cdot}31 - 2}{4{\cdot}31}\right) \times 12 - 2}$$

$$c = \frac{1200000}{43{\cdot}1 \times 10{\cdot}57 + 287{\cdot}4 \times \left(\frac{2{\cdot}31}{4{\cdot}31}\right) \times 10}$$

$$c = \frac{1200000}{455{\cdot}567 + 1540{\cdot}4} = c = \frac{1200000}{1995{\cdot}967}$$

$$c = 601 \text{ lb.}$$

This is quite satisfactory, and t and c_s can be proceeded with.

$$t = c\,m\left(\frac{d - n}{n}\right) = t = 601 \times 15\left(\frac{12 - 4{\cdot}31}{4{\cdot}31}\right)$$

$$t = 601 \times 15 \times 1{\cdot}784 = t = 16082 \text{ lb.}$$

$$c_s = c\,m\left(\frac{n - d_c}{n}\right) = c_s = 601 \times 15 \times \left(\frac{4{\cdot}31 - 2}{4{\cdot}31}\right)$$

$$c_s = 601 \times 15 \times \left(\frac{2{\cdot}31}{4{\cdot}31}\right) = c_s = 4832 \text{ lb.}$$

All these figures are quite satisfactory, and the beam can be considered as being calculated for the centre and end sections.

The diagram in Fig. 128 shows the section at the ends of the span. As the amount of reinforcement to take the compression at the ends was 9·58 in., it will be necessary to employ, say, six 1½-in. diameter rods, which will give a total area of 10·5 sq. in., and these must be placed in two rows as shown. The depth taken in the calculations will be to the centre of the two rows, and consequently, it will be necessary to allow 3 in. of concrete below this point, giving a total depth of 15 in. instead of 14 in., which was theoretically necessary. This increase will have the effect of making the resistance slightly in excess of that calculated, and therefore it may be adopted. The extra inch

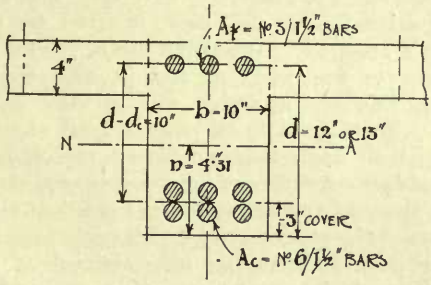


Fig. 128.—Section, at Ends of Span, of Tee Beam Fixed at Ends

of covering is not indicated on the section illustrated in Fig. 127, as it is not necessary, and the under side of the beam could be formed with a very slight camber, which would improve rather than detract from the appearance of the beam. The question of bending up certain bars from the lower surface to the upper near the supports, and the calculations for shearing stress, are not considered here.

TEE BEAMS WITH DOUBLE REINFORCEMENT

With regard to tee beams with double reinforcement, it is not considered necessary to give any particular examples for this class of calculation, as the same formulæ and principles will apply as were given for beams with double reinforcement when the neutral axis falls within the slab or coincides with the under side, and, as previously stated, these are the most common cases. It must be borne in mind, however, that b will represent the width of the flange acting with the beam in compression. When the axis falls below the slab, the beam can be designed upon the assumption that the concrete in the rib is to be neglected, and the various

formulæ previously given can be used, and if necessary slight adjustments can be made after the preliminary calculations have been made. Where double reinforcement is employed, the two sets should always be connected by binding.

SHEARING STRESS AND ADHESION

The question of the shearing stress and adhesion has been omitted up to the present in order to simplify matters and enable the student to become quite familiar with the principles of designing for tensional and compressional stresses; but the provision of steel in practically all concrete beams to resist shear is absolutely necessary, and this question has, in the past, received far too little attention. When the use of reinforced concrete was more or less in its infancy, this side of the design was so little dealt with that in nearly all cases of tests applied to beams where failure occurred, such failure was due to insufficient provision against shear or diagonal tension, and it was this fact that caused the more careful designers to take up this part of the subject more seriously.

The draft regulations of the London County Council make provision for shear members as follows: "All beams shall be provided with adequate shear members, and such shear members shall—(*a*) Be spaced according to the distribution and intensity of the shearing stresses; but the distance from centre to centre of the shear reinforcement at any part of the beam shall not exceed the effective depth of the beam. (*b*) At least extend from the centre of the tensile reinforcement to the centre of pressure in the concrete under compression. (*c*) Be passed under or round the tensile reinforcement, or be otherwise secured thereto. (*d*) Have a mechanical anchorage at both ends, or they shall have a mechanical bond with the concrete throughout their length."

The first point for consideration will be that dealing with the intensity and distribution of the shearing stresses. The shearing tendency is due to the opposition of the weight and the reactions, and as the greatest opposition will occur at the supports where the total weight meets, as it were, the total reaction, it is at this point that the greatest shearing stress is found, and the stress at any intermediate point will equal the reaction at the support, minus any portion of the load situated between this point and the support under consideration. Thus the amount of shearing stress is affected by the amount and nature of the load, but not by the span; and, consequently, it is in the case of beams which carry heavy loads over a short span that the shear becomes the most important consideration. The shearing stress acts both horizontally and vertically, and at any point in the beam these two actions will be equal, and as a result the shear members are often placed at an angle of 45 degrees to resist the resultant of the two forces. The distribution of the stresses over the area of the beam is somewhat curious, as the combined action of the bending moment and shearing force causes the lines of maximum stress to assume a curved form; and the shearing stress will be greatest where the tension and compression are nil, namely at the neutral axis.

Diagonal tension is often expressed as shear, whereas it is dependent on the shear and the longitudinal tension for its intensity. The actual moment of this intensity cannot be ascertained with any certainty, and, consequently, it is usual in practice to calculate the vertical shear, and take this as the measure of the diagonal tension, or, at least, provide sufficient steel for this purpose, when the beam should be efficiently designed. The shearing stress, as previously stated, is not uniformly distributed over the area of the section, but is greatest where the longitudinal stresses are least, namely, at the neutral axis, and in an ordinary homogeneous beam the stress diminishes from the axis to the outer edges, where it becomes nil, assuming a parabolic curve, as shown in diagram A (Fig. 129). If S equals the total shear at a vertical section, then the maximum shear which occurs at N A, and is expressed by $m\,s$, can be found as follows:—The area of the shaded portion represents the total shear on the section = S, and the area of this figure $= \frac{2}{3}\,m\,s \times d$, therefore $\frac{2}{3}\,m\,s \times d = \mathrm{S}$ and $\frac{2}{3}\,m\,s = \frac{\mathrm{S}}{d}$, or $m\,s = \frac{3\,\mathrm{S}}{2\,d}$. If a diagram is set up as given at A (Fig. 129), then the actual stress at any point in the section can be determined.

It is unimportant in most cases to know the exact distribution of the stress, but this instance is given rather to make the theory more explicit.

Now, in the case of a reinforced concrete beam, the distribution of the shearing stress is somewhat different, and the reason for

this is the fact that the concrete above the axis only is called upon to resist longitudinal stress, that below the axis being neglected, and all the tension is taken as coming upon the steel. The result of this assumption is that, although the shear above the neutral axis will gradually diminish towards the outer edge, below this point it will be uniformly distributed over the section as shown in diagram B (Fig. 129). As the upper part of this figure gives a parabolic curve, the equivalent area over which the stress acts above the neutral axis will equal two-thirds $b\,n$, and below the axis $b \times (d - n)$,

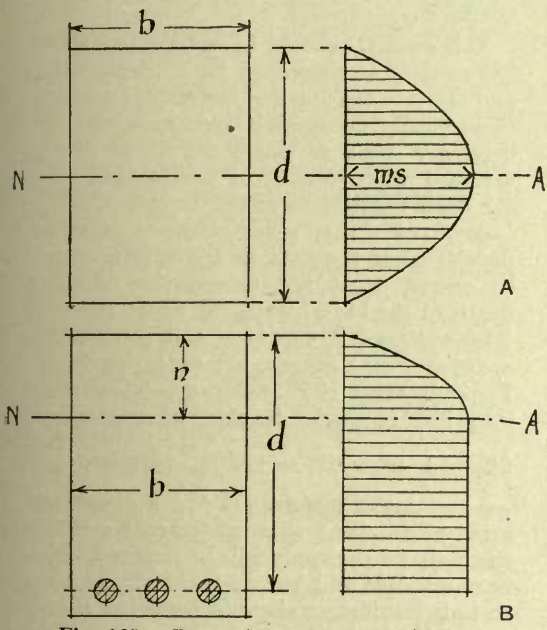


Fig. 129.—Determining Stress in Ordinary and in Reinforced Concrete Beams

therefore the total equivalent area, taking the shear, will equal $b \times \left(d - \frac{n}{3}\right)$. Where S equals the total shearing stress acting over the section, the greatest stress per square inch will be equal to $\frac{S}{b \times \left(d - \frac{n}{3}\right)}$ and this is expressed by the symbol s.

In designing the shear members, it is necessary to consider the amount of stress that is acting over a portion of the section equal to 1 in. in depth, and with a width equal to b, and this value will equal $\frac{S}{d - \frac{n}{3}}$ or $b \times s$, where b = the width of the section, and s = the maximum shear per square inch. In dealing with the calculations for shearing stress it is also necessary to consider that of the adhesion between the steel and the concrete, as the two materials can only act together when this adhesion is not overcome. The permissible allowance for concrete in shear has been given in the table of working stresses as 60 lb. per square inch, while the allowance for adhesion between steel and concrete is given as 100 lb. per square inch, and the shearing stress across the section divided by the total circumference of the rods or shear members must not exceed this amount. If the perimeter or circumference of the bars be expressed by O, then the intensity of stress per square inch around the bars will be equal to $\frac{S}{O\left(d - \frac{n}{3}\right)}$. In the designing of shear members, it is usual and advisable to utilise stirrups or rods of one section in the same beam and vary the spacing according to the amount of stress to be resisted. As the greatest shear will always occur at the abutment, the stirrups will be fairly close together here, and will gradually increase in spacing towards the point at which they are not required. To resist the vertical shear, there are the concrete and the horizontal reinforcement, but in the case of the horizontal shear it will in most cases be necessary to provide vertical members, as mentioned above, or crank up some of the tension rods at a suitable point, or provide both, and it will be necessary presently to illustrate the method of determining the positions of the shear members to suit the varying shearing stress.

To illustrate the use of the formulæ already given, examples may be taken and their application shown. In the case of ordinary slabs and rectangular beams with single reinforcement which are accurately calculated to resist the longitudinal stresses, it will be found that sufficient concrete is provided in the section to withstand the shearing stress, and, consequently, no special shear members are theoretically necessary, but for practical reasons they should be provided, as they assist in holding the concrete together, and, furthermore, they will be required under the regulations of the authority under whose jurisdiction the building will come.

As an illustration, consider a beam sup-

ported at the ends, which is required to carry a uniformly distributed load of 3,000 lb. per foot run, including the weight of the beam itself, over an effective span of 16 ft. The total load will equal the load per foot run, multiplied by the span = 3,000 lb. × 16 ft. = 48000 lb. The bending moment $= \frac{w\,l}{8} = \frac{48000 \times 16 \times 12}{8}$ = 1,152,000 in.-lb. Calculating the economical size of d by formula, we have $d = \sqrt[3]{\frac{B\,M}{57}} = d = \sqrt[3]{\frac{1152000}{57}}$ $= d = \sqrt[3]{20210}$ = say 28 in. The breadth should equal ·6 d, therefore b = ·6 × 28 in. = 16·8 in., say 17 in. A_t = ·00675, $b\,d$ = ·00675 × 17 × 28 = 3·213 sq. in. The greatest shear stress, as previously stated, will occur at the supports, where it will be equal to the reaction. This value will be $\frac{W}{2} = \frac{48000}{2}$ = 24000 lb. The load being uniformly distributed, the shear will diminish from the support to the centre of the span, where it will be nil, and, consequently, if calculations are made for the position where the greatest shear occurs, and the beam is found to be sufficient, it will be obvious that it will be safe at all points. The greatest shear is represented by S, and this equals 24,000 lb., while the formula for finding the maximum stress per square inch has been given as $s = \frac{S}{b \times \left(d - \frac{n}{3}\right)}$, where s = the maximum stress per square inch, which must not exceed 60 lb., or shear members will be theoretically required. Working this out, we get $s = \frac{24000}{17 \times \left(28 - \frac{10{\cdot}08}{3}\right)} = s = \frac{24000}{17 \times 24{\cdot}64}$ $= \frac{24000}{418{\cdot}88}$ = 57·5 lb., which is below the permissible limit. The value of n was taken at ·36 d, as the economical percentage of reinforcement was used, therefore n = ·36 × 28 = 10·08 in. as given.

The adhesion or shearing stress intensity round the reinforcing bars will now be considered; it must not exceed 100 lb. per square inch. The amount of steel required was found to be 3·213 sq. in., and if bars having a diameter of 1⅛ in. are used, the area of which equals ·994 sq. in., four of these would give an area of ·994 × 4 = 3·976 sq. in., which is more than sufficient. To test the adhesion, the formula $\frac{S}{O\left(d - \frac{n}{3}\right)}$ must be used. The circumference of one 1⅛-in. bar = 3·5343 sq. in., and the total circumference of the four bars = 3·5343 × 4 = 14·1372 sq. in. Then the intensity per square inch = $\frac{24000}{14{\cdot}1372 \times \left(28 - \frac{10{\cdot}08}{3}\right)} = \frac{24000}{14{\cdot}1372 \times 24{\cdot}64}$ $= \frac{24000}{348{\cdot}3406}$ = 68·8 lb., which is well below the limit.

This example should be sufficient to show the method of procedure for a simple case, and indicate the truth of the statement that theoretically no special shear members are necessary with a single reinforced beam designed with economical proportions. As the draft regulations of the London County Council state that shear members must be provided, and the distance apart, from centre to centre, must not exceed the effective depth of the beam, we could adopt stirrups 1 in. wide and $\frac{1}{16}$ in. thick, and place same apart at some distance, not exceeding 28 in. The span being 16 ft., this can be divided up into the requisite number of spaces, as follows: 16 × 12 = 192 in., therefore $\frac{192}{28}$ = say, seven spaces. If the stirrups are arranged to give a space at the centre of the span, six stirrups only will be required. The next example will be one in which the concrete is insufficient alone to resist the shear, and special shear members are therefore theoretically necessary.

As a further example, let it be required to design a beam to carry a uniformly distributed load of 2 tons per foot run over an effective span of 16 ft., such load to be inclusive of the weight of the beam itself, which is limited in size to 20 in. effective depth and 12 in. wide. Total load = 2 × 2240 × 16 = 71680 lb. $B\,M = \frac{w\,l}{8}$ if beam is supported only, therefore $B\,M = \frac{71680 \times 16 \times 12}{8}$ = 1720320 in.-lb. The size of the beam = 20 in. × 12 in., and the economical percentage, if singly reinforced, would be 20 × 12 × ·00675 = 1·62 sq. in. Next find the moment of resistance of such a beam, and if necessary provide additional

reinforcement to take the excess bending moment, as explained in doubly reinforced beams. $MR = BM = A_t \times t \times \left(d - \frac{n}{3}\right)$, and $n = \cdot 36 \times 20 = 7\cdot 2$ in., therefore $BM = 1\cdot 62 \times 16000 \times \left(20 - \frac{7\cdot 2}{3}\right) = 456192$ in.-lb.

The actual $BM = 1720320$ in.-lb., therefore excess $BM = 1720320 - 456192$ in.-lb. $= 1264128$ in.-lb., and this must be provided for by compressional and additional tensional reinforcement. Then

$$A_t^1 = \frac{\text{Excess } BM}{t \times (d - d_c)} = A_t^1 = \frac{1264128}{16000 \times (20 - 2)}$$

$$A_t^1 = \frac{1264128}{288000} = 4\cdot 38 \text{ sq. in.}$$

This amount must be added to the value given above of 1·62 sq. in., making a total amount for the tensional reinforcement of 6·00 sq. in. The compressive reinforcement will be found by the formula :

$$A_c = \frac{\text{Excess } BM}{c_s \times (d - d_c)}, \text{ where } c_s = \frac{t \times (n - d_c)}{d - n}$$

$$\text{equals } c_s = \frac{16000 \times (7\cdot 2 - 2)}{20 - 7\cdot 2} = c_s = \frac{83200}{12\cdot 8}$$

$c_s = 6500$ lb. Therefore,

$$A_c = \frac{1264128}{6500 \times (20 - 2)} = A_c = \frac{1264128}{117000}$$

$A_c = 10\cdot 8$ sq. in.

There is no need to explain the method of checking the longitudinal stresses, as that has already been done in previous examples, but the provision for shear may now be investigated. The total load is equal to 71,680 lb., and the greatest shearing stress will occur at the supports, where it will equal $\frac{W}{2} = \frac{71680}{2} = 35840$ lb. The allowance for shear on the concrete must not exceed 60 lb. per square inch, and therefore the value of the section to resist the shear will be equal to the equivalent shear area, which $= b\left(d - \frac{n}{3}\right) \times 60$ lb., therefore resistance of concrete $= 12 \times \left(20 - \frac{7\cdot 2}{3}\right) \times 60 = 12 \times 17\cdot 6 \times 60 = 12672$ lb. This value being less than the actual shear coming upon the section, it is obvious that there will be an excess equal to 35,840 − 12,672 lb. = 23,168 lb., which must be met by steel shear members. The working stress for steel in shear is given as 12,000 lb. per square inch, and the area of steel required will equal $\frac{23168}{12000} = 1\cdot 93$ sq. in. As the beam is supported at the ends only, and the tensional reinforcement at the centre of the span equals 6·00 sq. in., it will be clear that as the bending moment diminishes towards the supports the tensional reinforcement may be reduced, and consequently one or more of the bars may be cranked at a point where the shear on the concrete becomes over the permissible limit and carried through the shear area in such manner that it takes up

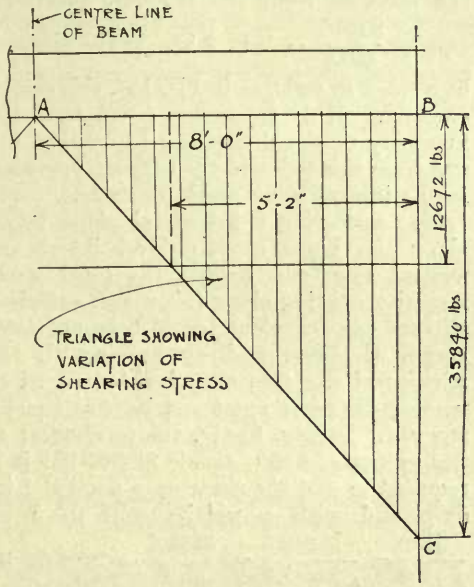


Fig. 130.—Vertical Shear in Reinforced Concrete Beam carrying Uniformly Distributed Load

the excess shear. Reference to Fig. 130 will show how the shearing stress decreases from the maximum value of 35,840 lb. at the support to nil at the centre, the diagram illustrating one half of the beam only, and consequently the total shear on one half the beam is represented by the triangle A B C. The shearing value of the concrete in section is 12,672 lb., and if this amount is set down to scale from B, as indicated, and a horizontal line is drawn through to the line A C, then the position where it cuts this line, as shown by the dotted vertical line, will be the point at which the shearing stress becomes greater than the concrete alone is capable of taking.

This point may be found by calculation as follows : The shear at any point is equal

to the load at the support minus the load situated between that point and the support. Now the load at the support = 35,840 lb., and as the value of the sectional concrete to take the shear is only 12,672 lb., it is necessary to move outwards from the support along the beam until sufficient load has been placed between us and the support to reduce the load to this amount. It will be necessary, therefore, to move outwards until an amount of 35,840 − 12,672 = 23,168 lb. has been passed over. The load is equal to 2 tons, or 4,480 lb. per ft. run, and the distance from the support, therefore, equals $\frac{23168}{4480}$ = 5 ft. 2 in. If the diagram be set out to scale, as in Fig. 130, this can be checked. It is from this point that the necessary tensional bars can be cranked up, and from this point to the support provision for horizontal shear must be made.

On considering a horizontal plane taken along the beam at any level below the neutral axis, and finding the total stress intensity existing along this plane, sufficient stirrups can be calculated to take the total excess of shear, and by keeping all the stirrups of one size, they can be spaced at varying distances apart and be thus equally stressed. Taking, firstly, the total shear at the supports, which equals 35,840 lb., it is required to find the shear on a portion 1 in. deep and with a width equal to b by

$$\frac{S}{d - \frac{n}{3}} = \frac{35840}{20 - \frac{7{\cdot}2}{3}} = \frac{35840}{17{\cdot}6} = 2036{\cdot}36 \text{ lb.}$$

Of this amount the concrete is capable of taking an amount equal to the width of the section multiplied by 60 lb., therefore value of concrete = 12 × 60 = 720 lb., leaving an excess of 2,036·36 − 720 = 1,316·36 lb. Now this excess, which has to be taken by the steel, will gradually diminish from the support towards the centre of the beam until it reaches a point 5 ft. 2 in. from the support, when it will be nil, as the concrete alone is capable of taking all the shear from this point to the centre of the beam. This is clearly shown in the diagram in Fig. 131. Having now obtained the maximum excess which requires to be taken by the steel, and the length of beam over which this is acting, the total excess may be determined by taking the mean of the excess at the support and at a point 5 ft. 2 in. from the support, and multiplying same by the length over which it is acting This amount will obviously be equal to the area of the triangle shown as D E F in Fig. 131, which equals

$$\frac{DE \times DF}{2} = \frac{1316{\cdot}36 \times 62}{2} = 40807{\cdot}16$$

in.-lb. This is the total horizontal shear, therefore, for which steel must be provided, and as the allowance for steel in shear is given as 12,000 lb. per square inch, the sectional area required will equal $\frac{40807}{12000}$ = 3·4 sq. in. If it is decided to use stirrups 1 in. wide and $\frac{1}{4}$ in. thick the sectional area would equal ·25 sq. in., and as each stirrup will have two

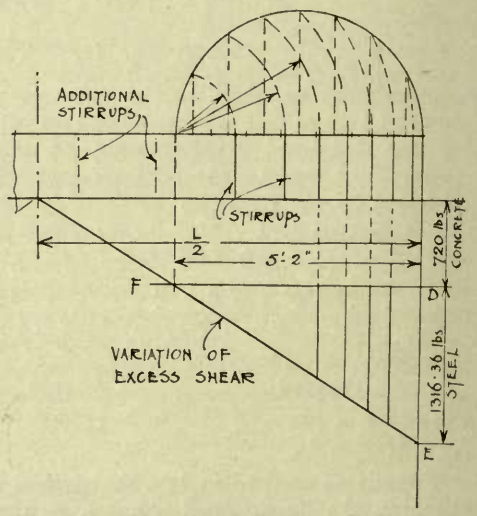


Fig. 131.—Horizontal Shear in Reinforced Concrete Beam carrying Uniformly Distributed Load

wings, the area provided by each stirrup will equal ·5 sq. in.

The total number of stirrups required will therefore equal $\frac{3{\cdot}4}{{\cdot}5}$ = say 7, and these must be spaced to give a uniform stress on each one. This may be done graphically, as illustrated in Fig. 131, where a semicircle is set up over the length of beam in which the stirrups are to be placed, and the length is also divided up into fourteen equal divisions, being twice the number of stirrups to be employed. Perpendicular lines are then erected to cut the semicircle as shown, beginning at one end and setting up a line from the first division mark, and then from every alternate mark until the opposite end is reached, and the last line erected over the point marking the end divi-

sion. The points at which these perpendicular lines cut the semicircle are then transferred to the horizontal line, marking the top of the beam with the varying radii as shown, when the positions of the stirrups will be obtained. The object of thus spacing the stirrups is, of course, to divide up the shearing stress represented by the triangle D E F into equal portions, and if the lines representing the stirrups are continued down to cut the triangle as indicated, it will be seen that it is divided up into divisions of varying width which have more or less equal areas.

To comply with the draft regulations of the London County Council it would be necessary to introduce the additional stirrups shown by dotted lines. These stirrups are spaced 20 in. apart, equal to the effective depth of the beam, and should have the ends turned out to afford a good bond with the concrete. In the case of inclined stirrups the best angle for these will be 45°, and it is necessary that they should be firmly attached to the horizontal rods if they do not actually form a part of same. The spacing for these can be found in a similar manner to that employed for vertical stirrups, except that the inclined line must be drawn through the point at which the vertical division cuts the neutral axis. There is no doubt that the best method is that where vertical stirrups are employed together with a certain number of bent-up bars, the method being effective in the resistance of diagonal tension.

COLUMNS

The theory and calculations in the case of columns must be divided into two distinct sections, namely, short columns and long columns, the reasons for dealing with compression members in this manner having previously been given in the notes on columns in the preceding chapter. In the case of reinforced concrete work the column may be considered as belonging to the first section where the length does not exceed eighteen times the least diameter, and as such short columns are considered to fail by direct crushing only, the full working stress may be adopted, and the design is comparatively simple. These will be considered in the first instance after a few general remarks relating to reinforced concrete as a material for columns.

The steel is usually placed in the form of vertical bars spaced equidistant on the circumference in the case of a circular column, and at each corner in the case of a rectangular section, except in the case of rectangular sections where the sides are unequal as given below; and these vertical bars must be linked or bound together with some form of lateral reinforcement to prevent the possibility of the load causing the bars to burst outwards through the comparatively thin covering of concrete coming over them. The steel rods themselves assist the concrete core when under the load, and prevent the bursting tendency, and to do this effectively they must be well connected, and by links which are close together, thus forming practically a steel cage which confines

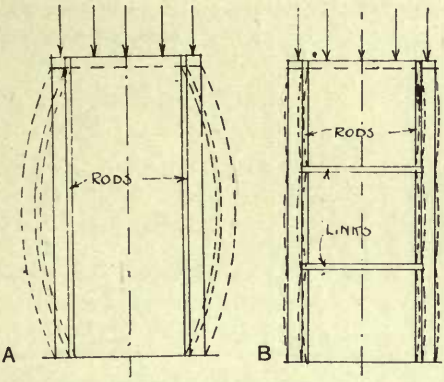


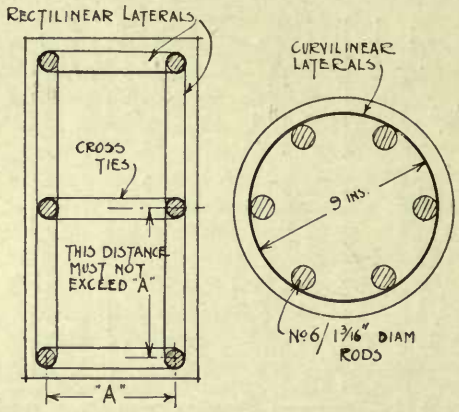
Fig. 132.—Diagrams Showing need for Lateral Reinforcement in Columns

the concrete and allows it to develop its full strength. It will be readily understood that in the case of a concrete column with vertical rods only there will be a great tendency for the rods to act as shown in an exaggerated manner by dotted lines in diagram A (Fig. 132), whereas, supposing two intermediate links were introduced, this tendency would be lessened, and the rods could only bend as shown in diagram B. If sufficient intermediate links are introduced the tendency will be overcome altogether, and it will be obvious, therefore, that the lateral reinforcement is an important factor in the design.

The draft regulations of the London County Council state that the total cross sectional area of the vertical reinforcement shall not be less than 0·8 per cent. of the area of the hooped core, and the volume of lateral reinforcement shall not be less than 0·5 per cent. of the volume of the hooped

core. The effective diameter must be measured from the outside of the outermost vertical reinforcement, and it must be measured in the direction of the lateral supports which determine the length of the pillar.

When the column is such that the laterals are rectilinear, there must be at least four lines of vertical reinforcement throughout the entire length, and if the laterals are curvilinear, there must be at least six lines of vertical reinforcement, and the diameter of the rods in any case must not be less than ½ in., and the pitch of the laterals must not exceed six-tenths of the effective diameter of the pier. In the case where rectangular piers are used in which the ratio between the greater and lesser diameter exceeds



Figs. 133 and 134.—Rectangular and Circular Columns with Respectively Rectilinear and Curvilinear Laterals.

one and a half, the cross section of the pillar must be sub-divided by cross ties, and the number of vertical rods shall be such that the distance between the rods along the longer side of the rectangle shall not exceed the distance between the rods on the shorter side. This is illustrated in Fig. 133, where the laterals are rectilinear, and the diagram in Fig. 134 gives an illustration of the use of curvilinear laterals. In the case of concrete in columns in simple compression the limiting stress is 600 lb. per square inch, and as the steel and concrete are considered as being shortened in the same proportion, the stress on the steel can never exceed m times the stress on the concrete, or, in other words, $m \times c$. As m equals 15, then the permissible value to allow for the steel equals $600 \times 15 = 9{,}000$ lb. per square inch. In the case of columns which are axially loaded, therefore, and when the height does not exceed eighteen times the least diameter, the safe load will equal the net sectional area of the concrete in the core multiplied by c plus the sectional area of the longitudinal reinforcement multiplied by $m \times c$. As an example, let it be required to find the safe load that can be carried by a rectangular column 15 ft. high which has an effective size of 12 in. by 12 in., and is reinforced with four vertical bars, each having a diameter of 1 in.

The area of one rod will equal ·7854 sq. in., and the total area of steel will equal $4 \times \cdot 7854 = 3{\cdot}1416$ sq. in. The net area of the concrete will equal $(12 \times 12) - 3{\cdot}1416 = 144 - 3{\cdot}1416 = 140{\cdot}8584$ sq. in. The safe load on the column will equal $A \times c + A_v \times m\,c$, where A equals the effective area of the concrete, and A_v equals the sectional area of the steel. Working this out gives $140{\cdot}8584 \times 600 + 3{\cdot}1416 \times 15 \times 600 = 84515{\cdot}04 + 28274{\cdot}4 = 112789{\cdot}44$ lb., which equals just over 50 tons. If preferred, the total area of steel and concrete can be taken and multiplied by c, and to this the extra value of the steel can be added by taking the sectional area and multiplying same by fourteen times c. Having already considered the area of steel as taking 600 lb. per square inch, it will be clear that the extra value will equal $9{,}000 - 600$ lb. $= 8{,}400$, which is the same as fourteen times c. Let the previous example be worked out in this manner. Then the safe load $= 144$ sq. in. $\times 600 + 3{\cdot}1416 \times 14 \times 600 = 86400 + 26389{\cdot}44 = 112789{\cdot}44$ lb. as before. To state this as a formula, then W the weight $= A \times c + A_v \times 14\,c$, or $A_v \times 14\,c = W - (A \times c)$, therefore $A_v = \dfrac{W - (A \times c)}{14\,c}$.

This last formula will give the method to be adopted when designing a short column to carry a certain given load, an example of which is shown. Design a reinforced concrete circular column which is 12 ft. high to carry an axial load of 40 tons. First settle the diameter, which must be at least one-eighteenth of the height. If we take 9 in., this will be one-sixteenth, and therefore well within the limit. The sectional area will equal $9 \times 9 \times \cdot 7854 = 63{\cdot}6174$ sq. in., and the load equals $40 \times 2240 = 89{,}600$ lb.

Then $A_v = \dfrac{89600 - (63{\cdot}6174 \times 600)}{14 \times 600} =$

$A_v = \dfrac{51429{\cdot}56}{8400} = A_v = 6{\cdot}12$ sq. in. If six

rods are used, the area of each one will require to be 1·02 sq. in. The area of a rod having a diameter of $1\frac{3}{16}$ in. equals 1·1075 sq. in., and this size will therefore do very well, and the total area of the steel will equal 1·1075 × 6 = 6·645 sq. in. For the purpose of practice, check the column and see what safe load it will carry, bearing in mind that the practical area of the steel equals 6·645 sq. in. as against the theoretical area of 6·12 sq. in., and the safe load should therefore be slightly over 40 tons.

The safe load equals $A \times c + A_v \times m\,c$, therefore $W = \{[(9 \times 9 \times \cdot 7854) - 6\cdot 645] \times 600\} + (6\cdot 645 \times 15 \times 600) = W = \{[63\cdot 6174 - 6\cdot 645] \times 600\} + (6\cdot 645 \times 9000)$

$W = 34183\cdot 44 + 59805$

$W = 93988\cdot 44$ lb. $= 41\cdot 5$ tons.

Which is correct. The column will then be as shown in Fig. 134.

Having dealt with short columns, it now remains to deal with those known as long columns, namely, where the effective diameter is less than one-eighteenth of the free length. In such columns, as previously stated, there is a tendency for the column to fail by bending, although, theoretically, there should be no such tendency if the column is absolutely axially loaded; but such loading cannot be absolutely guaranteed in practice, and experiments that have been conducted from time to time with long columns go to show that it is not advisable to load the column to its full capacity as though failure would occur through direct crushing. This bending tendency cannot be measured or calculated unless a certain definite amount of eccentricity is known, and consequently an empirical formula has to be used where certain constants are given deduced by experiment.

It is interesting to note that the London County Council draft regulations contain conditions which limit the loading on long columns, and if the designer is content to accept the rules laid down by such regulations, he will have no need to employ a complicated formula. The conditions are given, as they will no doubt be of some assistance, and particularly in the case of checking the carrying capacity of a column already designed. Briefly stated, the rules are as follows:—A pillar shall be deemed to have fixed ends when the ends of the pillar are sufficiently secured to other parts of the construction, having such rigidity as will maintain the axis of the pillar at the ends in its original vertical position under all loads less than the crippling load. When both ends are fixed, and when the ratio of length to the effective diameter does not exceed—

18	The full stress may be allowed on the pillars
21	·8 of above may be allowed
24	·6 ,, ,, ,,
27	·4 ,, ,, ,,
30	·2 ,, ,, ,,

For other ratios the stress shall be proportionate to the above. In the case of compression members not having both ends fixed, the loads shall be as follows: Let P = the maximum pressure on pillars and compression members having fixed ends.

Condition of Ends	*Working load*
One end fixed and one end hinged	$\frac{P}{2}$
Both ends hinged	$\frac{P}{4}$
One end fixed and the other end free and not guided, stayed or supported in all directions ..	$\frac{P}{16}$

Unfortunately, there is a great difference between the various formulæ that have been deduced by various authors, and if the student were calculating, say, a wooden strut, and he adopted Gordon's formulæ, and checked the result by using Ritter's formula, he would find that the latter would give a load which would be about five times as great as that found by Gordon's. Thus, there is no real proof as to what safe load can be adopted, or such a variation would not be possible, and the most that can be done is to recommend the student to adopt a fairly safe method. Gordon's formula has already been given in the preceding chapter; it is one of the best known in this country, and is greatly used. In the absence of extensive experiments with long, reinforced concrete columns, this formula may be adapted for designing such columns, as hereafter explained. The formula is as follows: $R_c = \dfrac{A\,r_c}{1 + a\dfrac{l^2}{d^2}}$, where R_c = the safe load in the same units as r_c, A = the area of the

cross section, r_c = the safe resistance of the material in compression, a is a constant derived by experiment, l is the length in the same units as d, and d is the effective diameter. It will be seen that the value $A\,r_c$ would equal the safe resistance if the column were a short one, and consequently the value $1 + a\frac{l^2}{d^2}$ is the reducing factor which is dependent on the ratio of length to diameter, and also upon a, which is governed by the conditions of fixing. In the table given on p. 79 it will be seen that for wrought-iron and steel, where the section was rectangular or circular, the values for a were as follow: Ends hinged $\frac{4}{2500}$; ends fixed $\frac{1}{2500}$, and one hinged and one fixed $\frac{1}{1000}$, and these same constants may be taken. As the value $A\,r_c$ in this formula equals the safe resistance of a short column, this can be substituted by the expression which is already deduced for giving this value, namely, $A \times c + A_v \times m\,c$, and if the reducing factors given above are applied, the complete formula becomes

$$W = \frac{A \times c + A_v \times m\,c}{1 + a\frac{l^2}{d^2}}$$

and this must be used when checking the weight-carrying capacity of a column which has a diameter less than one-eighteenth of the free length.

In the case of designing a short column, it was shown that the area of steel required was calculated by the formula $A_v = \frac{W - (A \times c)}{14\,c}$, where A = the total area of the cross section. It will now be obvious that the value $1 \times a\frac{l^2}{d^2}$ must again be introduced, and more steel will be required than would be the case with a short column. The actual weight must therefore be multiplied by this value, giving a formula as follows:

$$A_v = \frac{W \times \left(1 + a\frac{l^2}{d^2}\right) - (A \times c)}{14\,c}.$$

Having now deduced formulæ for finding the area of steel required, and also for checking the column to see the safe load which it can carry, the application of these will be shown in an example. A column is fixed at the ends, and has a free length of 20 ft., while the load to be carried is 50 tons. The column is to be rectangular, and it is limited in size to 12 in. square. Calculate the amount of vertical reinforcement required, and afterwards check the load the column as designed would carry. The size given is the effective size, and the load is assumed to include the weight of the column itself. The load equals 50 tons, which is equivalent to 112,000 lb. The effective sectional area will equal $12 \times 12 = 144$ sq. in., and as the ends are fixed, the value of a will be $\frac{1}{2500}$. The steel will be found by the formula:

$$A_v = \frac{W \times \left(1 + a\frac{l^2}{d^2}\right) - (A \times c)}{14\,c}, \text{ therefore}$$

$$A_v = \frac{112000 \times \left(1 + \frac{1}{2500} \times \frac{240^2}{12^2}\right) - (144 \times 600)}{14 \times 600}$$

$$A_v = \frac{112000 \times (1 + \cdot 16) - 86400}{8400}$$

$$A_v = \frac{129920 - 86400}{8400} = A_v = \frac{43520}{8400}$$

$A_v = 5{\cdot}18$ sq. in.

If four bars are used, then each will require an area of 1·295 sq. in. A round bar with a diameter of $1\frac{5}{16}$ in. has an area of 1·353 sq. in., and this is the nearest size obtainable, and the total area of steel required will therefore be $1{\cdot}353 \times 4 = 5{\cdot}412$ sq. in. As designed, therefore, the column is 20 ft. high, fixed at the ends, 12 in. square effective size. and reinforced with four $1\frac{5}{16}$ in. diameter bars. This should next be checked by calculating the safe load it will carry, and, to be correct, this load must not be less than 50 tons. Now

$$W = \frac{A \times c + A_v \times m\,c}{1 + a\frac{l^2}{d^2}}, \text{ therefore}$$

$$W = \frac{138{\cdot}588 \times 600 + 5{\cdot}412 \times 15 \times 600}{1 + \frac{1}{2500} \times \frac{240^2}{12^2}}$$

$$W = \frac{83152{\cdot}8 + 48708}{1{\cdot}16} = W = \frac{131860{\cdot}8}{1{\cdot}16}$$

W = 113673 lb. = 50·7 tons, which is quite satisfactory, the slight amount over 50 tons being due to the fact that the actual amount of steel equals 5·412 sq. in., which is slightly in excess of the theoretical amount calculated, namely, 5·18 sq. in.

In order to form a comparison with the

conditions given in the draft regulations of the London County Council, as previously stated, it will be interesting to calculate what load could be carried by this column if the rules given therein are applied. In the first instance, it will be necessary to work out the load that could safely be put on a short column having the same sectional area and reinforcement, and then apply the reducing factor which is dependent on the ratio of diameter to length, as given in the table previously.

The safe load for a short column would be found by the formula:

$W = A \times c + A_v \times m\,c$, therefore

$W = 138{\cdot}588 \times 600 + 5{\cdot}412 \times 15 \times 600$

$W = 83152{\cdot}8 + 48708 = W = 131860{\cdot}8$ lb.

This, then, would equal the safe load for a short column, and the ratio of diameter to length must next be considered. The length is given as 20 ft., and the effective diameter equals 12 in., and the ratio, therefore, equals 20. Upon referring to the table, it will be seen that where the ratio of diameter and length does not exceed 21, the safe load can be taken as ·8 of that for a short column. The safe load for this example will therefore equal 131860·8 lb. × ·8, which equals 105488·64 lb., or 47·09 tons. This is slightly less than the safe load, as calculated by the formula, which was 50·7 tons and when working in actual practice, it would be necessary to keep within the limits allowed by the London County Council rules, if within the County of London. It must be remembered, also, that the ratio of length to diameter is actually 20, and not 21 as taken, and this, obviously, will affect the result. It is very seldom, however, that such a fine line would be drawn, and the designer in calculating the loads c nnot work so closely to the actual loads as to give no margin whatever. As stated previously, the various formulæ and methods for calculating compression members will always be found to vary considerably, and in this instance there is only a difference of about 6 per cent.

COLUMNS ECCENTRICALLY LOADED

It is proposed to give a few notes on the eccentric loading of columns and its effect, although the value of the eccentricity is very difficult to obtain, unless such eccentricity is caused by certain definite loads applied at a certain definite distance from the centre of the column. Eccentricity in loading is caused in many ways, a common case being that where the load from a beam is transmitted through a bracket projecting from the column, or by the beam not being carried right over the column. The eccentricity is sometimes such that it causes one edge of the column to be in tension, and sufficient reinforcement must then be provided to resist same. The fact of the load not being axial will cause the stress in the column to vary over the area of the cross section, and the compression will be at its maximum at that edge which is the nearest to the eccentric load, and at a minimum at that edge which is the farthest away from same.

It is necessary, therefore, to consider the two extremes, and this can be done by finding the pressure that would occur if the load were axial, and adding or deducting the value of the eccentricity. Where the compression is at a maximum, therefore,

$$\text{Max. } c = \frac{W}{A_E} + \frac{W\,y}{Z}$$

where W = the load, A_E = the equivalent area of the concrete in square inches = the sectional area of the concrete + 14 times the sectional area of the steel $= A + (m - 1)\,A_v$, y = the eccentricity of the load in inches, or, in other words, the distance from its point of application to the neutral axis of the section, and Z = the section modulus. This last expression was explained in the preceding chapter, and the following values for reinforced concrete columns are given.

$$\text{Rectangular } Z = \tfrac{1}{6}\,A\,d + \tfrac{1}{2}\,(m - 1)\,A_v\,\frac{d_1^{\,2}}{d},$$

where A = the total area of the cross section, d = the outside diameter of the column, and d_1 = the distance from centre to centre of the vertical reinforcement. Circular column with four bars only

$$Z = \tfrac{1}{8}\,A\,d + \tfrac{1}{2}\,(m - 1)\,A_v\,\frac{d_1^{\,2}}{d},$$

and for circular column with bars arranged in a circle,

$$Z = \tfrac{1}{8}\,A\,d + \tfrac{1}{4}\,(m - 1)\,A_v\,\frac{d_1^{\,2}}{d}.$$

On that edge farthest from the load the minimum compression $= \text{Min. } c = \dfrac{W}{A_E} - \dfrac{W\,y}{Z}$.

From the above formula may be deduced a formula for finding W, which is

$$W = \frac{\text{Max. } c \times A_E \times Z}{Z + A_E \times y}.$$

It will now be advisable to work out an example to show the application of these

formulæ. Find the safe load that can be carried by a column 16 in. square reinforced with eight $1\frac{1}{4}$-in. rods, the load being applied vertically at a distance of 2 in. from the axis, and the distance from centre to centre of

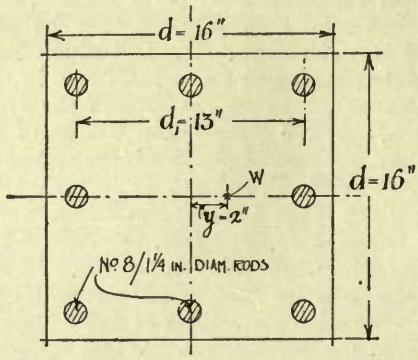


Fig. 135. Designing Column Eccentrically Loaded

rods being 13 in. The maximum stress on the concrete is not to exceed 500 lb. per square inch. The plan of the column, as given, is shown in Fig. 135, and upon reference to this the symbols can be followed. The safe load will be found according to the formula :

$$W = \frac{\text{Max. } c \times A_E \times Z}{Z + A_E \times y}.$$

Max. c is given as 500 lb., and $y = 2$ in., while A_E will equal $A + (m - 1) A_v = 16 \times 16 + (15 - 1) 9{\cdot}8176 = 256 + 137{\cdot}446 = A_E = 393{\cdot}446$. A_v is given by the area of the rods, the area of one $1\frac{1}{4}$-in. rod $= 1{\cdot}2272$ sq. in., and the total area $= 1{\cdot}2272 \times 8 = 9{\cdot}8176$ sq. in., as given above. The value of Z must next be found by the expression

$$Z = (\tfrac{1}{6} A \times d) + \tfrac{1}{2} (m - 1) A_v \times \frac{d_1^2}{d},$$

therefore,

$$Z = (\tfrac{1}{6} \times 256 \times 16) + \tfrac{1}{2} \times 14 \times 9{\cdot}8176 \times \frac{13^2}{16}$$

$$Z = 682{\cdot}66 + (68{\cdot}7232 \times 10{\cdot}56)$$

$$Z = 682{\cdot}66 + 725{\cdot}717$$

$$Z = 1408{\cdot}377.$$

Having determined the value of the various factors, proceed to find the value of the weight as follows :—

$$W = \frac{500 \times 393{\cdot}446 \times 1408{\cdot}377}{1408{\cdot}377 + 393{\cdot}446 \times 2}$$

$$W = \frac{277060148{\cdot}571}{2195{\cdot}269}$$

$W = 126207$ lb., or just over 56 tons.

As a check to this, and also as an example of the use of the formulæ for the maximum and minimum compression, let it be required to calculate the maximum and minimum compression on the column above mentioned, and carrying the calculated eccentric load. Then the maximum compression, which should not exceed 500 lb., will be found by the formula :

$$\text{Max. } c = \frac{W}{A_E} + \frac{W y}{Z}.$$

The values of A_E and Z have already been calculated, and therefore,

$$\text{Max. } c = \frac{126207}{393{\cdot}446} + \frac{126207 \times 2}{1408{\cdot}377}$$

$$\text{Max. } c = 320 + 179{\cdot}2 = 499{\cdot}2 \text{ lb.},$$

which is within the limit. The minimum compression will be given by the expression :

$$\text{Min. } c = \frac{W}{A_E} - \frac{W y}{Z}$$

$$\text{Min. } c = \frac{126207}{393{\cdot}446} - \frac{126207 \times 2}{1408{\cdot}377}$$

$$\text{Min. } c = 320 - 179{\cdot}2 = 140{\cdot}8 \text{ lb.}$$

The edge of the column farthest from the load is therefore not in tension, and the column is satisfactory. The diagram given in Fig. 136 shows how the stress varies across the section, and the length of the vertical line at any intermediate point represents the

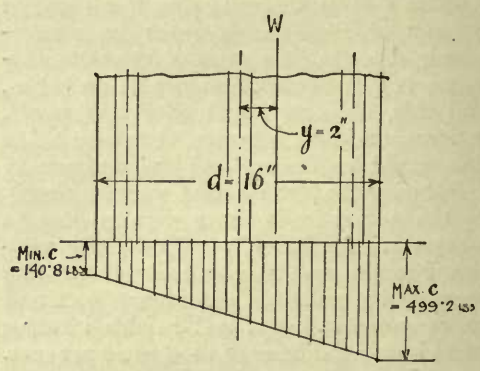


Fig. 136.—Stress in Column Eccentrically Loaded

compression stress at that point. The stress on the steel being fifteen times that on the concrete the value can be found in any of the bars by scaling the length of the line coincident with the axis of the bar, and multiplying same by 15.

RETAINING WALLS

It is not proposed in these notes to cover the whole theory and design of retaining walls, but rather to give some general remarks to enable the student to form an idea of the principles governing the design of such structures. Walls may be used to retain or support either earth or water, and the method employed in reinforced concrete is very different from that which is customary with brickwork masonry or plain concrete.

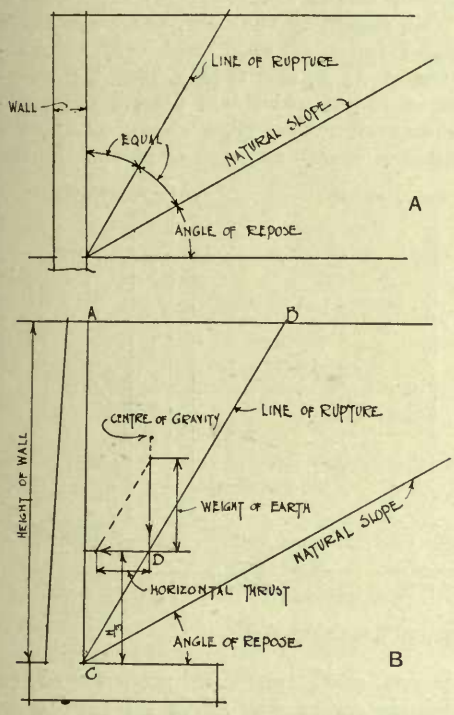


Fig. 137.—Influence of Angle of Repose and Line of Rupture on Design of Retaining Walls

In the last-mentioned case the wall is dependent on its thickness and weight, which must be such that its gravity is not overcome by the pressure acting on the back of the wall; whereas in the case of reinforced concrete the wall is designed to offer sufficient resistance by its strength. Tension at the back of a reinforced concrete wall is permissible, and steel is provided to resist it; but it is obvious that a brick wall, for example, would be capable of resisting very little tensional stress. A good comparison of the two methods of design is offered by the front and back retaining walls constructed at the Royal Automobile Club (described in a later chapter), where both plain and reinforced concrete were used.

In dealing with walls to retain earth, it is necessary, in the first instance, to investigate the nature and value of the force that has to be resisted. It will be quite clear that a mass of earth standing alone without lateral support will not remain with vertical faces, but that some of its particles will fall past one another, since they have little or no cohesion, and form a mound having sloping surfaces. Sand poured upon a floor forms a mound having sloping sides, and it will be seen that there is a certain angle with the horizontal which the material will maintain even without lateral support. The angle depends on the friction between the particles, and is called the "angle of repose"; it varies in different materials, and also in the same material according to the state of consolidation and dryness. The following table must be taken merely as a guide, and cannot be considered as fixed by any hard-and-fast rule:—

Material	*Angle of Repose*
Wet clay or vegetable earth ..	16°
Wet sand	20°
Dry clay, sand, or vegetable earth	30°
Sandy gravel	26°
Clean, firm gravel	45°
Loose shingle	40°
Hard, dry vegetable earth ..	45°
Clay, well drained	45°

This angle can be considered, then, as the natural angle at which the material will rest, and this is shown in diagram A (Fig. 137), where the line of rupture is also indicated. It is considered that only the wedge-shaped portion of earth contained between the line of rupture and the face of the wall will require to be supported, as upon the sudden removal of the wall it is that portion which would slip, and this has been proved more or less by actual experience. There will also be a large amount of friction set up between the portion that is tending to slide and the portion that is normally at rest, and by considering that portion above the line of rupture only, an allowance is made for this, and it will be seen in the diagram that the wedge-shaped portion in tending to slip downward is really sliding on the inclined plane represented by the line of rupture.

The line of rupture is found by setting out the angle of repose and bi-secting the

angle between this and the back of the wall; the earth that is contained in this triangle is calculated and the load worked out according to the weight per cubic foot of the particular earth under consideration. It is usual to take a length of 1 ft. in the calculations, both in calculating the weight of the soil to be supported and in the design of the wall. In the calculations it is necessary to consider both the vertical portion of the wall and also the base, which requires to be sufficiently large to distribute the pressure over such an area of foundation as to bring the load per square foot within the safe resistance of the soil. Also the base must be of sufficient thickness to allow the projecting portions to act as cantilevers. The vertical portion will be a cantilever having a length equal to the height of the wall, and it is obvious that the greatest stress will occur at the bottom, and this will be due to the outward pressure of the earth which can be ascertained as shown in diagram B (Fig. 137), when the centre of pressure of the triangular prism of earth A B C is shown at a point equal to $\frac{h}{3}$ from the bottom, where h equals the height of the wall. The value of the pressure will be found by calculating the weight of the earth contained in the triangle, and also the centre of gravity of the triangle as illustrated. The weight will act downward through this centre of gravity, and if a line is drawn downward through this point until it cuts the line of rupture at D as shown, and the amount of the weight is set up to scale from this point, the horizontal equivalent thrust on the back of the wall will be found by drawing a line parallel to the line of rupture from the top of the line representing the weight until it cuts the horizontal line drawn through D. This horizontal thrust can then be multiplied by its distance from the point of intersection with the base, which will equal $\frac{h}{3}$, and this will give the bending moment to be resisted. The bending moment will gradually diminish towards the top, where it will be nil.

With regard to the base, there are three general types of wall employed, as illustrated at A, B, and C (Fig. 138), the selection of one of which will depend upon circumstances. The type shown at A, for example, could not be employed if the adjacent earth was the property of an adjoining owner who might raise objection to the projection under his land. This was the case at the Royal Automobile Club, and it became necessary to design the wall with the projecting toe extending into the building as illustrated at B. In the former case the weight of the earth acting downward on to the base will tend to prevent the overturning of the wall, whereas in B this is not the case, and sufficient weight must be provided to prevent the wall from being thrust over on the extreme outer point of the toe, which will be the fulcrum of the lever. The type shown at C is somewhat between the others, having a projection on both sides. Provision to prevent sliding on the base is often made by stepping the under side in such manner that vertical surfaces are provided, as shown

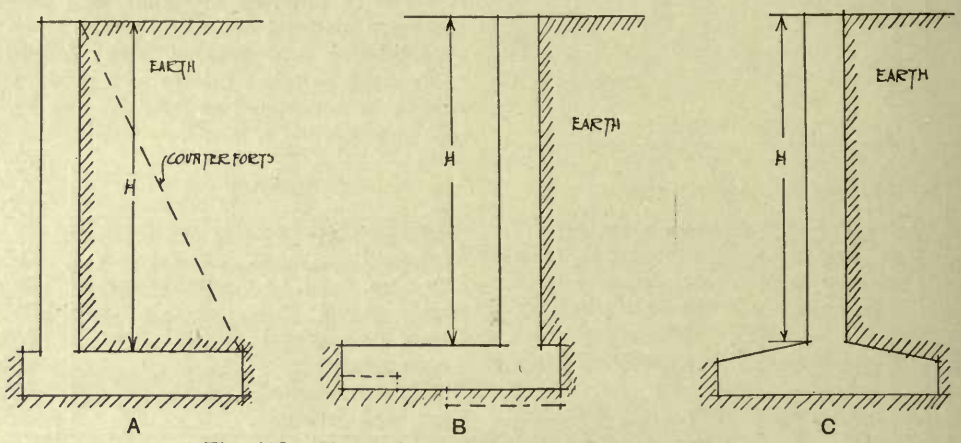


Fig. 138.—Three General Types of Retaining Walls.

by dotted lines in B. The length of the base for preliminary calculations is often taken as about one-half of the height, and the thickness is, of course, calculated from the bending moment, which it has to resist as a cantilever.

The type shown at A is often stiffened by counterforts at intervals, as shown by dotted lines, these serving as stiffeners, both to the vertical wall and the base. In London work it is often necessary to construct vaults under the pavement, when the outer

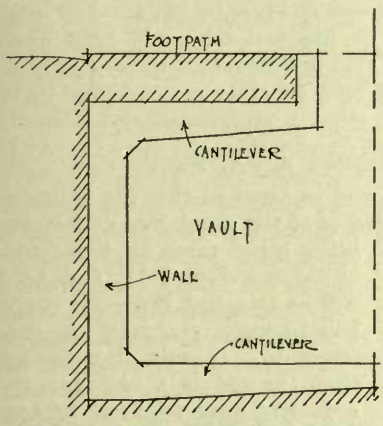


Fig. 139.—Retaining Wall with Cantilevers under Footpath

wall will act as a retaining wall to support the roadway. A type that is sometimes employed under such circumstances is illustrated in Fig. 139, where the top horizontal portion acts as a cantilever from the vertical wall and supports the footpath, while the base acts as a cantilever from the bottom of the wall and has to resist the earth pressure. This type of construction was employed at Messrs. Whiteleys' new building in Queen's Road, Bayswater, London.

In the case of walls called upon to resist the pressure of water, matters are somewhat simplified, as there is no variation due to weight or angle of repose, and the pressure will increase according to the depth of the water and always act at right angles to any surface. A cubic foot of water weighs $62\frac{1}{2}$ lb., and the pressure in pounds per foot against the wall will therefore equal the depth of the water in feet multiplied by $62\frac{1}{2}$. It will be seen, therefore, that the pressure varies directly as the depth as shown by the shaded portion in Fig. 140, and the centre of the whole pressure will be situated at the centre of gravity of the triangle, which will be situated at one-third the height from the bottom. The value of this pressure will equal the height multiplied by the mean pressure, the latter being $= \frac{62\frac{1}{2} \times \text{height}}{2}$. This pressure multiplied by the height from the intersection of the wall with the base will give the

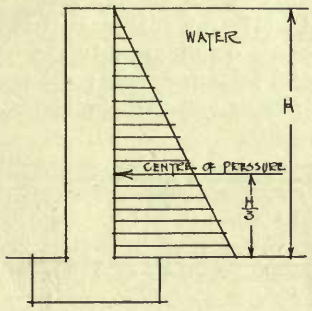


Fig. 140.—Retaining Wall to Resist Water Pressure

maximum bending moment. In the case of walls where the surface next the water is sloping, the total pressure will be greater or less according to whether the surface is sloping away from or to the water; but as the pressure will always act at right angles to the surface, the variation in the angle of the pressure will compensate for any variation in the amount of the pressure. The subject of retaining walls is one that requires very careful consideration when designing, and the student is advised to acquire a knowledge of trigonometry before attempting to become fully conversant with this branch of reinforced concrete work. A study of the examples already carried out in practice will also be helpful.

The Erection of a Reinforced Concrete Building

In this chapter will be described the method or system of carrying out the construction of the complete carcass of a reinforced concrete building. Attention will be drawn, not only to the correct way of doing the work, but also to the faults and pitfalls that have to be guarded against.

In reinforced concrete work there are four things that need to be most carefully observed : (1) strict adherence to the specialist engineer's drawings as to the deposition of the steel reinforcements ; (2) careful and efficient mixing and punning of the concrete so that the steel is entirely surrounded by this material ; (3) the proper design and the complete stability of the falsework ; (4) the proper and careful striking of the falsework.

On these items the strength, durability, and efficiency of the structure depend, and if they are not most strictly observed failure is almost inevitable. Failure, be it always remembered, not only means great and unnecessary expense, but involves enormous risk to life and limb. It need hardly be said that it behoves all those who undertake reinforced concrete construction to employ only reliable supervision and labour.

BUILDING A FACTORY

It will be assumed that the building to be erected is of the warehouse or factory type, being some three or four stories in height (*see* Fig. 141), and to which will be attached the usual engine and boiler-house, generally a one-story structure. It will also be assumed that the site is a bad one, and the engineers have found it necessary to provide a reinforced concrete pile foundation for the main building, and a reinforced concrete raft for the engine and boiler-house. To the warehouse there will be a large basement, which will involve the erection of a retaining wall to support the roadway on the frontage.

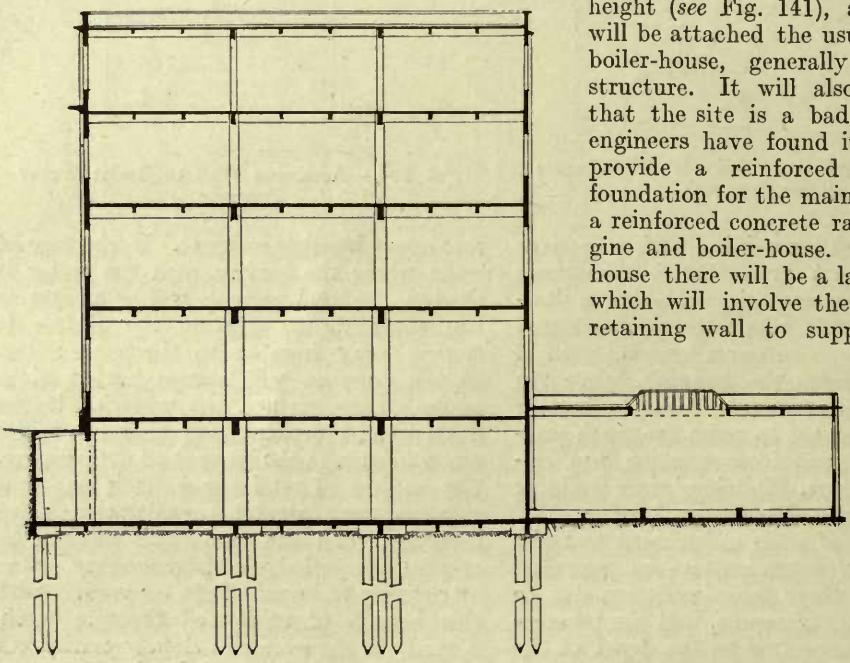


Fig. 141.—Vertical Cross Section of Typical Factory Building in Reinforced Concrete

In starting a job of this description, the first thing is to clear the site of all rubbish, hack up all old foundations, and leave the site clear for immediate operations. Next, the general foreman must select suitable positions for his office and the office of the clerk of works, and for the building material stores ; for the last-mentioned it will be necessary to erect some sheds. The contractor is now ready to receive materials.

Receiving the Bars.—Considering first the arrival of the steel reinforcement rods, these should be unloaded carefully so that the thinner rods are not twisted; they are easily put out of shape by rough handling. All the rods and bars must be sorted and bundled in their respective lengths and diameters, a complete list of which must be kept by the foreman. The sizes should be indicated by pegs or in some other way, so that the risk of errors may be lessened; the importance of this will be realised when it is remembered that a unit of one-sixteenth of an inch is adopted in specifying the bars, and should bars be in a wrong place or of a smaller diameter than detailed trouble may be caused. Bars that arrive twisted and distorted must be put aside to be straightened.

Steel bars are frequently laid down in the open, but this is not good practice, and temporary sheds should be erected for their reception, because exposed steel becomes badly scaled and pitted with rust, which ought not on any account to be permitted. Though it has been recognised that slight rust is beneficial for the protection of steel in concrete, deep pitting or rusting means the reduction of the diameter appreciably. The amount of rust permissible will occur while the steel is lying in a temporary building. Where there is no room for sheds, the steel should be given a coat of cement wash and then covered with tarpaulins. It is a good practice to erect temporary racks or stands in the sheds for the reception of the bars, so that they may be easily lifted down when required; this method will prevent excessive distortion in bars less than $\frac{5}{8}$ in. in diameter, which get very badly twisted when laid in rough heaps of considerable weight; the wholesale straightening of bars should be unnecessary. At the end of each division in the steel rack a board should be attached with the lengths and diameters of the rods indicated thereon, as in the accompanying table, the numbers in the first column indicating the number of bars in stock, and those in the second giving the lengths in feet and inches.

$\frac{3}{8}$ in.		$\frac{7}{16}$ in.		$1\frac{1}{8}$ in.	
50	5—0	120	10—0	32	14—0
50	5—6	128	12—6	38	16—6
35	6—9	141	26—6	58	17—9
42	9—6	163	27—6	64	18—6
82	11—6	181	30—0	73	19—0
95	20—6	200	31—6	80	20—0

By this simple means any bar of any particular length can be obtained at once, which is impossible when all bars of the same length are bundled together irrespective of their thickness.

Aggregate and Sand.—The position allotted for the deposit of the gravel and sand should be as near the mixing stage as possible, so as to save long barrow runs. In most cases the sand and gravel come to the job already screened and washed, and in a large contract this is nearly always advisable, as otherwise a gang of men would have to be kept continually washing and screening. Should the contractor be fortunate enough to find sand, etc., on the site, the washing, etc., will be inevitable. The gravel or aggregate must be deposited in a separate heap from the sand. Both the clerk of works and the foreman should see that there is no clay or loam mixed with the sand or gravel. Should broken stone or broken brick constitute the aggregate, see that it is free from dust; in the case of furnace slag or similar material, no sulphur or other impurities should be present. In regard to sand, sharp coarse river sand is preferable, or good pit sand of various size grains up to particles that will pass through a $\frac{1}{4}$-in. *square* mesh, and of which at least 75 per cent. will pass through a $\frac{1}{8}$-in. *square* mesh. Sand must be free from all ligneous (woody), organic or loamy substances.

Cement.—For the cement a specially dry shed should be erected. The familiar rough sheds, which are not proof against weather, should not be permitted. As to the testing of the cement, that is fully dealt with elsewhere in this book; but it should be said here that the foreman and the clerk of works should see that the cement arrives in sealed bags bearing the maker's name, and they should from time to time make practical tests to see that the cement is cool and in perfect condition. Cement must always be stored out of contact with wind, damp air, damp walls, or damp ground, as otherwise it quickly loses its strength.

Water.—If practicable, water should be supplied from a main, but if not, it should come from a good spring or boring. It is best to have the water laid on by a supply pipe to the mixing stage, and such a supply is a necessity when machine mixing—which is here advocated—is adopted.

Mixing Stage.—The foreman must consider carefully the position of the mixing stage or the mixing machine, as it is from this point that the building grows. Preferably the stage should always be open to

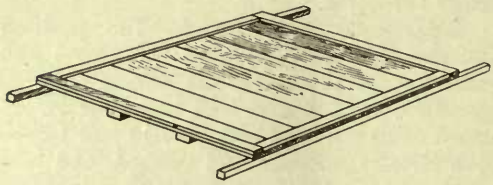


Fig. 142.—Mixing Stage

inspection throughout the entire period of the job; certainly it should not be hidden in an obscure corner, where it is difficult to watch and check the operations. The ideal position for a mixing stage is one where it can remain as long as possible and where it is convenient for the hoisting of the concrete to the different parts of the building. The matter is entirely one for the good judgment of the foreman. The mixing board, measuring about 15 ft. by 15 ft., should be constructed of good, straight, dry boards of even thickness, braced together at each end, and framed up on scantlings of 2 in. by 4 in., the longitudinal ones projecting at each end to form handles (*see* Fig. 142). A 2-in. fillet nailed round the board will prevent the wastage of cement. A level space should be prepared for the mixing board.

Hoisting.—Provision should be made for hoisting the concrete, timber and steel. On a large job a derrick and Scotchman should be erected, as well as a concrete hoist, some efficient types of which have been introduced.

TOOLS AND APPLIANCES

Concretors' tools include many that are familiar to the plasterer, while for finishing concrete surfaces he uses hacking tools (*see* p. 244), the idea of which has been borrowed from the mason's kit. The tamping or punning irons, for ensuring that the concrete is well tamped around the reinforcement, may be of the shapes shown in Figs. 143 to 146 and 152. Other punning tools, special spades, etc., are shown by Figs. 147 to 151.

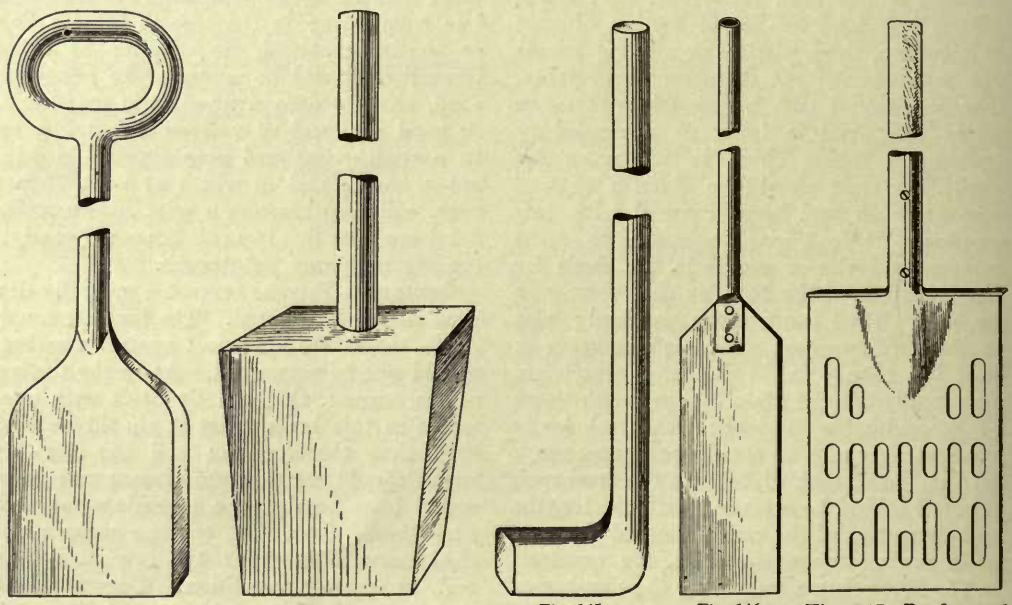


Fig. 143 Fig. 144 Fig. 145 Fig. 146 Fig. 147.-Perforated Spade

Figs. 143 to 146.—Four Patterns of Iron Tamper

For cutting bars, a hack saw—preferably a machine saw—is useful for large sizes; bars can also be severed by nicking and then breaking. Hand-operated shearing machines are available for rods of ordinary size. Small bars used as stirrups can be cut through with a hammer and chisel, with a

KEY TO THE NOTATION USED IN "CASSELL'S REINFORCED CONCRETE"

In Pillars and Beams

A = the effective *area* of the pillar.

A_c = *area* of *compressional* reinforcement (in sq. in.).

A_E = *area equivalent* to some given area or *area* of an *equivalent section* or equivalent area.

A_s = cross-sectional *area* of a vertical or diagonal *shear* member, or group of shear members, in the length p, where p = pitch of stirrups.

A_t = *area* of *tensile* reinforcement (in sq. in.).

A_v = *area* of *vertical* or longitudinal reinforcement in sq. in.

a = *arm* of the resisting moment or lever *arm* (in in.).

a' = *arm* ratio = $a'd$ ∴ $a'd = a$.

a_c = depth or distance of the centre of compression from the compressed edge.

B M = *bending* moment of the external loads and reactions (in lb. in.).

Generally, b = *breadth*.

In Tee Beams

b = *breadth* of flange of beam (in in.).

b_r = *breadth* of *rib* of T beam (in in.).

In Beams

c = *compressive* stress on the compressed edge of the concrete (in lb. per sq. in.).

C = total compression on the concrete (in lb. per sq. in.).

In Pillars

c = working *compressive* stress on the concrete of the hooped core.

c_s = *compressive stress* in the steel (in lb. per sq. in.).

In Circular Sections

Generally, d = *diameter*.

In Rectangular Sections

Generally, d = *depth*.

In Pillars

d = the *diameter* of the hooped core in in.

In Beams

d = effective *depth* of the beam (in in.).

d_c = *distance* from compressed edge to *centre* of compressional reinforcement.

d_s = total *depth* of the *slab* (in in.).

In Pillars

d_1 = *distance* between the centres of *vertical* bars measured perpendicular to the neutral axis.

E_c = *elastic* modulus of *concrete* (in lb. per sq. in.).

E_s = *elastic* modulus of *steel* (in lb. per sq. in.).

I = *inertia* moment of a member.

l = *length* of a pillar or effective length of span of beam or slab.

m = *modular* ratio = $\frac{E_s}{E_c}$.

M S = maximum shear.

n = *neutral* axis depth—i.e. depth of *neutral* axis from the extreme compressed edge (in in.).

n' = n/d = the *neutral* axis ratio ∴ $n'd = n$.

O = perimeter or circumference of bars.

P = total safe *pressure*.

In Pillars

p = the *pitch* of the laterals in in. (i.e. the axial spacing of the laterals).

In Shear Formulæ

p = *pitch* or distance apart (centre to centre) of the shear members or groups of shear members (measured horizontally).

In Beams

r = A/bd = *ratio* of area of tensile reinforcement to the area bd.

R^1 = left-hand reaction.

R^2 = right-hand reaction.

S = total *shear* in lb. at a vertical section.

s = intensity of the *shearing* stress on concrete in lb. per sq. in.

s_s = *shearing* stress on the *steel* (in units of force per unit of area).

T = total *tension* in the steel (in lb.).

t = *tensile* stress on the steel (in lb. per sq. in.).

In Pillars

V = *volume* of hooped core in cub. in.

W = total *working* load or *weight* on any member.

w = *weight* or load per unit of length of span.

y = *eccentricity* of the load measured from the centre of the pillar (in in.).

Z = section modulus.

In Gordon's formula:—R_c = total safe resistance to compression. r_c = total resistance of material to compression.

Note.—The above is based on, and modified from, the notation proposed by the Concrete Institute and adopted by the Royal Institute of British Architects, but certain symbols have been omitted, since they are not used in the book, and a few others have been added. The notation is built up on the principle of an index, the significant word in each term having been abbreviated to the initial letter, subscript letters being added in many cases. Capital letters indicate moments, areas, volumes, total forces, total loads, etc. Small letters indicate intensity of forces, intensity of loads, intensity of stresses, lineal dimensions (lengths, distances), etc.

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hammer and anvil block, as shown in Fig. 153, or with cutting pincers or pliers, several excellent patterns of which are now available.

Fig. 148. —Ross Spade

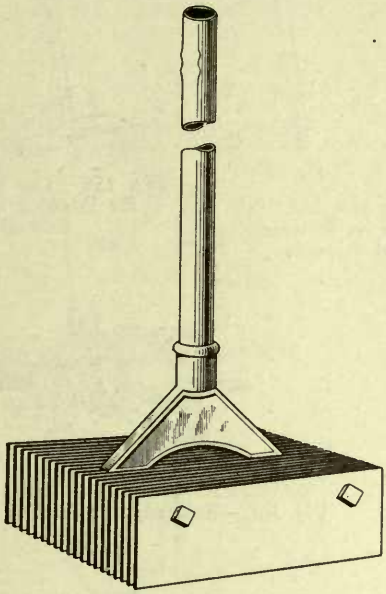


Fig. 149.—Tamper for Producing Fine Surface

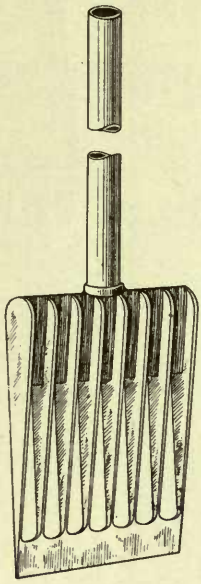


Fig. 150.—Special Spade for Facing

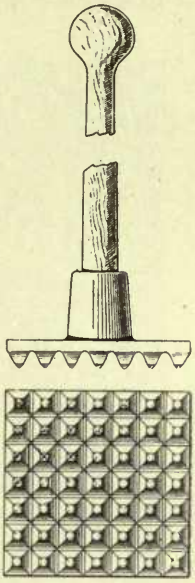


Fig. 151.—Andrews Tamper

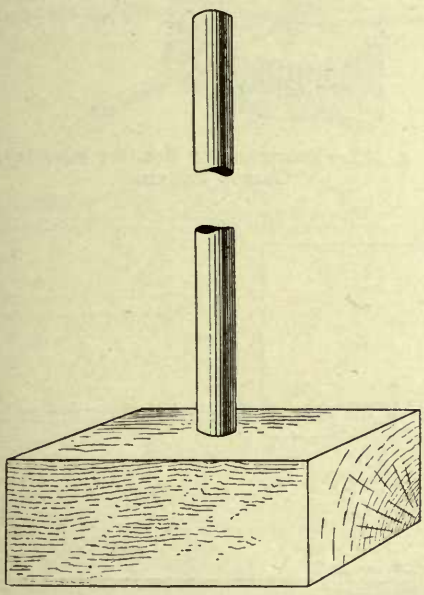


Fig. 152.—Wooden Tamper

Bending appliances include hand tools and machines. The wrenches shown by Figs. 154 and 155 are useful for general bending, especially in the case of bars partly embedded in concrete. The claw wrench (Fig. 156) is helpful for bending ends of stirrups over bars, and is believed to have been introduced by the Coignet firm. The wrench shown by Fig. 157 answers a similar purpose. By means of the key or twister shown by Fig. 158, the ends of stirrups can be twisted together. For the wiring together of the reinforcements where they intersect one another, practical workers have their own individual methods which they believe to be best, but there are undoubted advantages in employing a simple tool of the type introduced in the United States under the name of the Curry tyer. The Curry tyer is about 12 in. long, and the method of using it is shown in Fig 159. The wire ties used are a few inches long, the actual length depending upon the thickness of the bars,

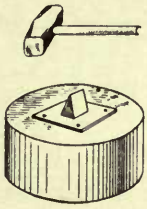


Fig. 153.—Cutting Anvil and Hammer

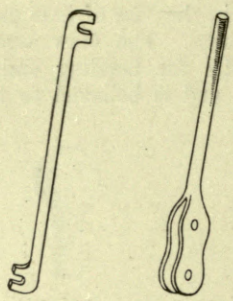


Fig. 154 Fig. 155

Figs. 154 and 155.—Wrenches for Bending Ends of Bars

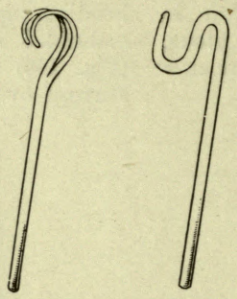


Fig. 156 Fig. 157

Figs. 156 and 157.—Wrenches for Bending Ends of Stirrups

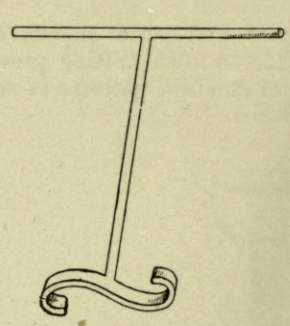


Fig. 158.—Key or Twister for Bending Ends of Stirrups

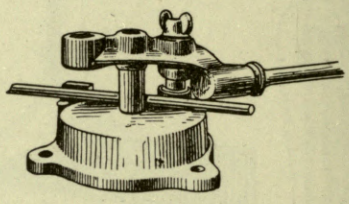


Fig. 161.—Kennedy Bar Bending Machine No. 1

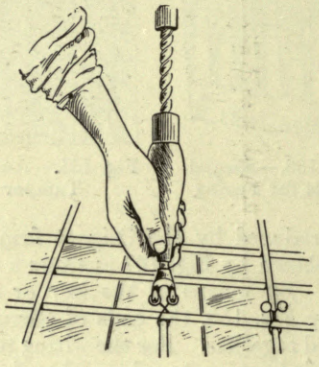


Fig. 159.—Curry Tyer

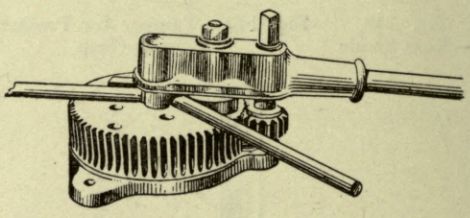


Fig. 162.—Kennedy Bar Bending Machine, Geared Pattern

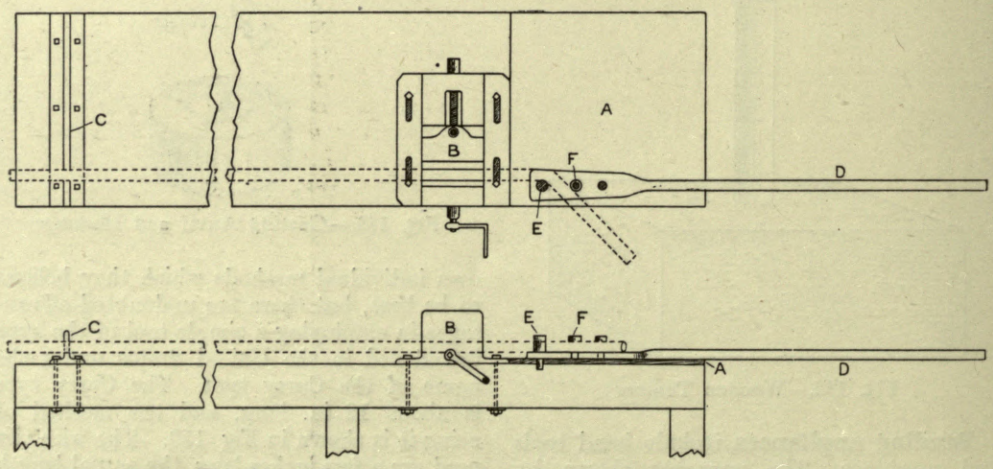


Fig. 160.—Elevation and Plan of Bench Bending Machine

and they have a loop at each end. The tool itself is a twisting appliance resembling in principle an automatic screwdriver, there being a sliding handle working up and down in a spiral groove machined on a revolving shaft inside the handle. The working end of the tool has two hooks; over one of these is passed one loop of a wire tie, the tie is then passed around the place of intersection, and the other loop is caught over the remaining hook. The sliding handle is then drawn back smartly, with the result

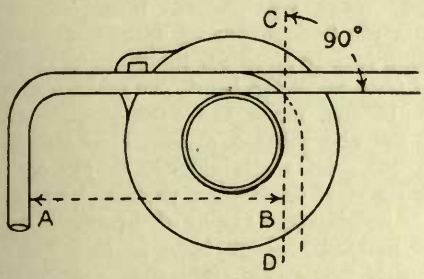


Fig. 163.—Making Bend to Given Inside Measurement

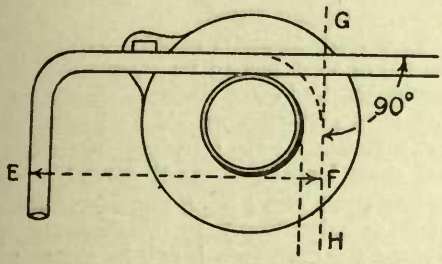


Fig. 164.—Making Bend to Given Outside Measurement

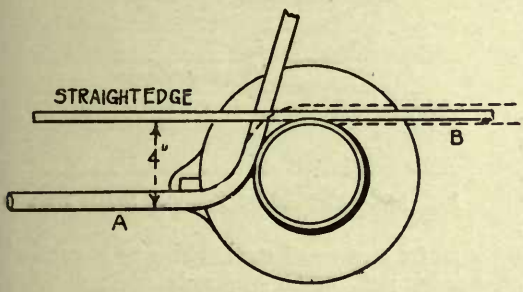


Fig. 165.—Making a Double Set

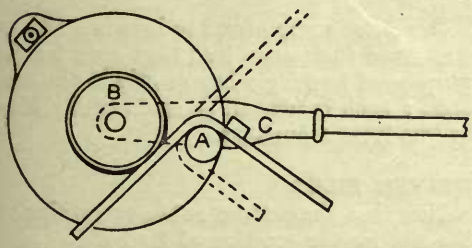


Fig. 166.—Making Sharp Bend in Thin Bar

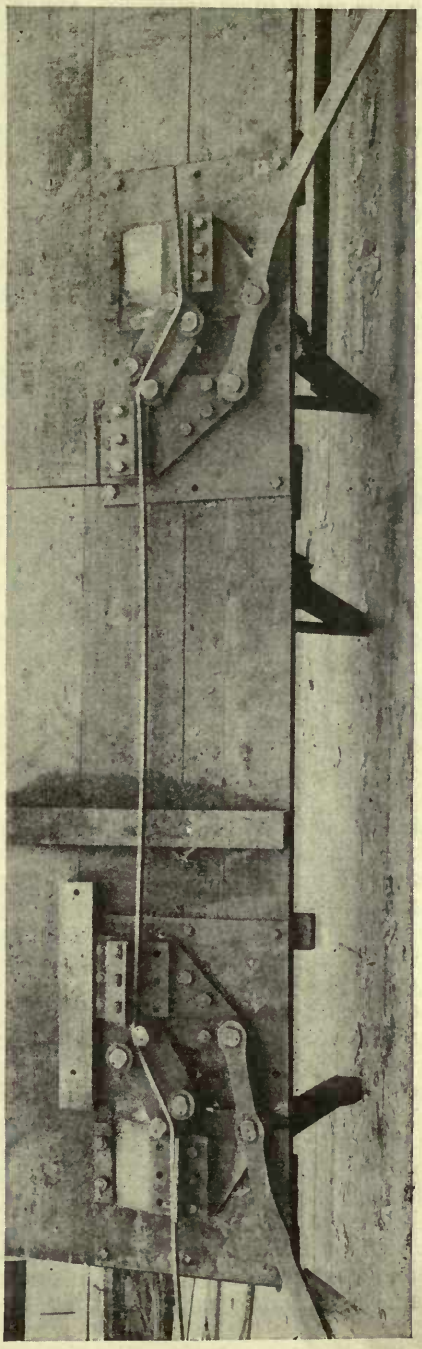


Fig. 167.—Photographic View of Right and Left Double Bar Bender

that the two ends of the tie are twisted together tightly. Ties can be made by this means at least twice as fast as when pliers and straight wire are used, besides which the work is more uniform.

In all cases it is advisable to bend bars cold; a course that is now easily possible with bars up to 1½ in. diameter. The small bars (up to 1 in. in diameter) can be bent on a long, strongly constructed, temporary bench to which is attached a vice for holding the bar firmly, the end to be bent lying between two steel pins fixed in a template on the bench; a strong steel lever with a hole in it to fit over one of the pins must be used, the bar lying across the lever between the steel pin on which the lever rotates and the pin fixed on the lever. By pulling the lever round, the bar is bent to any angle. Fig. 160 shows a machine of this type. To obtain the correct angle, a clear chalk line is marked on the bench to the angle required, and the bar is pulled over to lie on it. The machine above described can be made on any job at a cost of about 30s., and possibly less, as the contractor would probably have sufficient stock material at his workshops to enable him to fit it up. The bending bench shown by Fig. 160 and above referred to is based on one illustrated in the Coignet handbook. The rigid bench has a hard-wood top A, vice B, and bar rest C. The lever D rotates on a fixed pivot E, and has holes for pins or rollers F. Such a machine is suitable for bars up to 1¼ in. diameter. A right and left double bar bender on somewhat similar lines is shown in the photographic view (Fig. 167).

Another method of bending bars cold is by means of a special bending machine, and Kennedy's may be mentioned as being a most effective device by means of which cold bars up to 1½ in. in diameter may be bent.

The Kennedy bar-bending machine is made in four patterns. No. 1 (see Fig. 161) has a direct lever, and bends bars up to ⅝-in. diameter, the smallest radius being ½ in.; its weight is 25 lb. As shown, the bar is held between a stop and a central mandrel or former, the bending force being applied by a lever through a pulley in contact with the bar. No. 2 (see Fig. 162) is geared and will bend 1-in. bars to a ¾-in. radius; its weight is 124 lb. No. 3 is much the same as No. 2, but in addition there is a ratchet arrangement, while No. 4 is a powerful worm-geared machine for bending 1½-in. bars. To obtain a given measurement inside to inside of bends A B (Fig. 163), having made one bend, place the bar as shown, mark off distance required, mark the line C D square with A B, and then make the second bend. When a given measurement E F (Fig. 164) is to be taken over all, make the first bend, mark off desired measurement G along the bar, square off from G as before, taking care to allow for thickness of the bar, and then make the bend. To make a double set, having made the first bend, reverse the bar and place it against the stop, as in Fig. 165; let a straightedge be put against the mandrel or former, parallel with A, so as to indicate the extent of the second bend. For making a sharp bend in thin bars, an extra former A (Fig. 166), shaped to suit the section of the material, is inserted, the bar placed between A and the central former B, and the lever C, carrying a stop instead of a pulley, is pushed round.

When large diameter bars have to be bent, the aid of heat has to be sought, but they should not be heated above a cherry red—that is to say, they should be heated only just sufficient for the purpose. For stirrups and short lengths of small diameter bars, such as $\frac{3}{16}$ in. and ¼ in., a single bend can be made quickly simply by bending the bar round a steel pin on a bench by means of a wrench, as shown in Fig. 168; another way

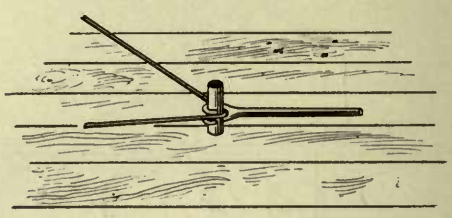


Fig. 168.—Method of Bending Small Bars

is to fix a piece of steel tubing to a vertical wooden post, nail a stop underneath it, place the bar in the tube, and pull it over, by which means stirrups can be made very quickly.

In almost every case, the bars in a slab or wall have their ends just bent over for the purpose of forming a key or grip in the concrete; such bends can be made with a blow or two from a hammer.

DRIVEN PILES

Now that the contractor has everything on the site for the erection of his building, the

pile making and driving is the first thing to which he must turn his attention.

The Pile Skeleton.—For the steel skeleton, the correct bars specified must be selected and placed on trestles close enough to prevent sagging; the bars are then placed in wood templates, which may be square or circular according to the engineer's requirements. These templates must be removed as soon as sufficient binding is done to hold the longitudinal bars in their correct positions. The next process is to add the binding, which may be done in several different manners according to the design of the pile. In the case of a square pile of the Hennebique type, links have to be made which wrap round two rods at a time, as clearly illustrated in Fig. 169. These links must be exactly spaced at the given pitch shown in the illustration. For a circular pile of the Coignet type (shown by Fig. 170) the links or ties encircle the bars, and the ends of the links are bent tightly round a single bar, and at the point where the links touch the bars they should be tightly tied with annealed wire. For a pile of the Considère type, which is octagonal in shape, the binding is of spiral form, the spiral being first wound on a drum and then threaded on to the bars. This type of pile is shown in Fig. 171. The pitch of the spiral must be strictly adhered to. Whilst these skeleton frameworks are being made, the joiners will be preparing the wooden moulds for their reception, a detailed description of which is given in the next chapter. As the moulds can be used and re-used, it is at the foreman's discretion how many he makes, but there should be sufficient to allow of the work continuing without hindrance; the more times one mould can be employed the greater the economy, and it will here be useful to state that in ordinary circumstances pile moulds can be removed in four or five days after the concreting. When several moulds are completed, the steel skeletons, which have been carefully lifted from the trestles and laid on planks on level ground, should be taken up and placed in the moulds, and the cast-iron pile shoe will then have to be fixed to the bottom of the steel framework in the mould. Having made a careful inspection to see that the steel framework is lying true in the mould, a gang of concreters with punning rods or rammers should be set to work; every bit of concrete placed in the mould should be carefully punned so that the steel is completely covered. Keen supervision should be exercised, because if by any chance any cavities form, or there is a large piece of aggregate loose in a pile, disaster will overtake that pile under the first blow of the driving-ram.

In very important work it is better to cast the piles in a vertical position, this being considered to give a more uniform strength over the cross section.

When the concreting is completed, the uppermost face must be carefully levelled

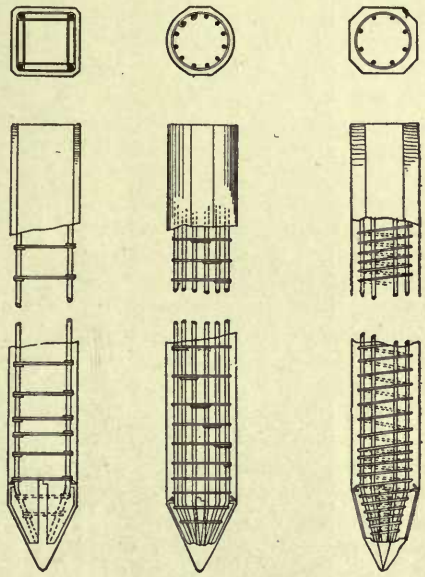


Fig. 169.—Hennebique Square Pile **Fig. 170.—Coignet Round Pile** **Fig. 171.—Considère Octagonal Pile**

off, and the pile may be left to dry. In hot weather the concrete should be watered (by means of an ordinary watering-pot fitted with a rose) daily for a week or ten days; but this should not be done until the concreting has been completed for ten or twelve hours. Ordinarily, a well-made pile can be safely driven in six to seven weeks after making. In cold weather the newly-made pile must be carefully protected from frost by covering with sacking, etc. When the piles have been in the moulds for four or five days, as already stated, the moulds may be struck—work that must be carefully done, so that the arrises are not chipped off and the steel bared. This

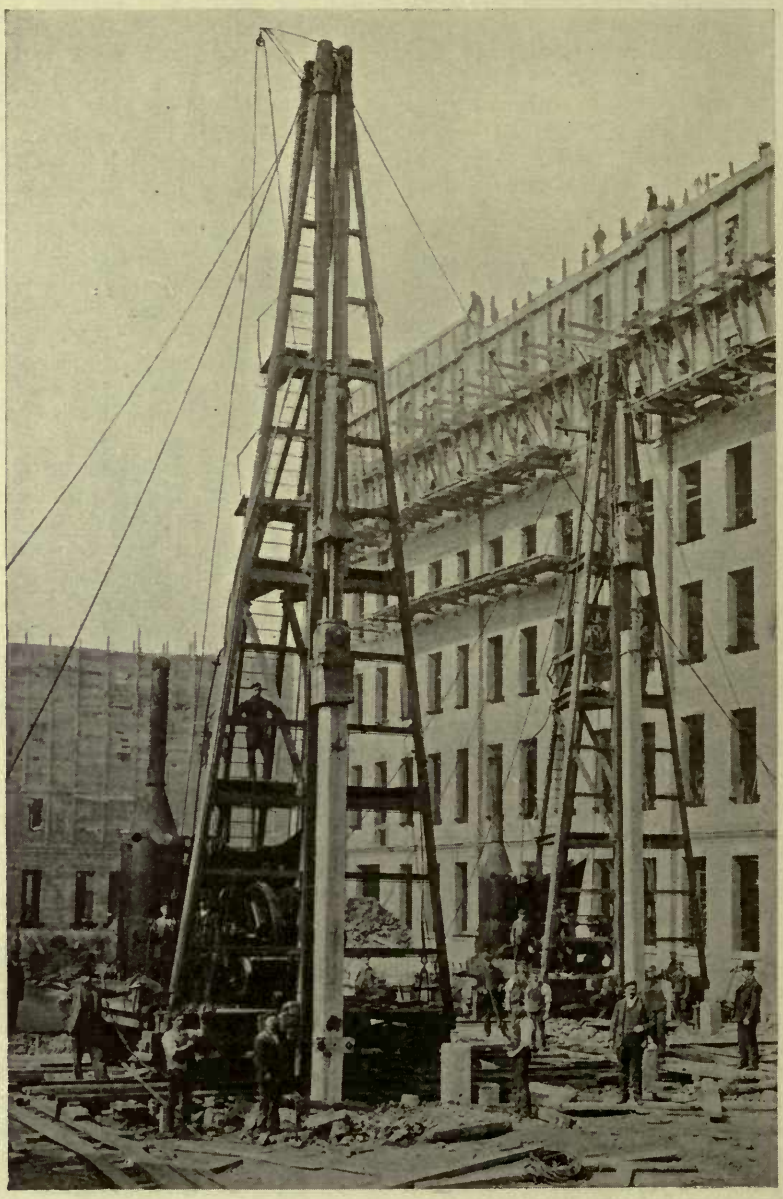


Fig. 172.—Lidgerwood Pile-driving Engine and Reinforced Concrete Pile

applies to circular as well as square and other piles, because the "circular" pile has two flat sides about 4 in. wide for the purpose of facilitating the driving, an arris being therefore formed at the juncture of the flat side and the circular side, as shown in Fig. 170. In cases where a wooden dolly alone is used without a cast-iron helmet on the head of a pile during driving, it is advisable when making the pile to use a richer mixture for a distance of from 2 ft. to 2 ft. 6 in. down from the head, the object being to strengthen the head so that it will not be seriously damaged under the impact (a wooden "dolly" actually takes the blow of the ram or monkey). The proportions of the concrete for use in piles need to be determined scientifically.

Driving of Piles.—Reinforced concrete piles are driven with a special pile-driving machine or engine, among the best known being the Lecour, Sykes, and Lidgerwood. The last-named is shown in Fig. 172. The drivers are generally fitted with a two-ton ram, which is hoisted by means of a steam winch. The ram can be driven at speeds up to thirty or forty blows a minute. Care and judgment must be exercised to drive with steady, uniform blows with a short drop, otherwise damage will be done by splintering and cracking the pile-head, however good and hard the concrete may be. With a trustworthy ganger driver who will take care piles can be driven without a crack. Owing to the great weight of a long pile, say one from 30 ft. to 50 ft. in length, special arrangements should be made for careful handling, hoisting, and placing in position for driving. When a pile is hoisted into its correct position, it must be plumbed up, bolted on—or rather through—the two leaders of the piling frame, and then plumbed up again when lowered before any blows are given. When a cast-iron helmet is used, as generally is the case with a Hennebique pile, a strong cushion of sawdust in sackcloth is packed inside the helmet to relieve the head of the pile from the severe impact caused by the fall of the ram, thus saving the head of the pile from shattering. When a helmet is not used, an elm block or dolly about 3 ft. long is placed on the head of the pile for the same purpose as the helmet. However, Considère piles, by reason of the special form of their reinforcing, are generally driven direct, without any helmet or dolly.

Everything being ready, the pile driver proceeds with the driving, which is continued until the specified set or stopping place is reached; the set is specified in the engineer's or architect's specification, and must always be strictly observed. The test required for the set of a pile usually is, that it does not sink more than ½ in. or ¼ in. under ten blows of a two-ton ram having a drop of 3 ft.; but much depends on the soil and the nature of the strata through which the pile has to penetrate.

It is desirable to note a few precautions that should be taken by the clerk of works in testing a pile set. The general practice is first to determine the distance of the drop of the ram to the head of the pile; this is done by clearly marking on the piling frame by means of a chalk line the height to which the ram is to be hoisted for each blow. For marking the pile, a gauge rod is hung from the leaders of the piling frame in such a manner that it may swing. A line is drawn on the pile at the level reached by the bottom of the gauge. The specified number of blows is then given, and the pile is again marked with a pencil at the bottom of the gauge rod. The space made between the line drawn on the pile before the delivery of the blows and the line drawn afterwards determines the going or sinking of the pile under the blows.

In witnessing a test, the clerk of works or inspector should observe the following points very minutely:—(1) When a helmet is used for capping a pile, see that the sawdust cushion packed inside is the same one that has been used during the driving of the pile, and that another has not been substituted, or that a new one has not been packed into the helmet and the old one placed under it so that on looking up the inspector sees the old cushion apparently undisturbed. Should a new cushion be inserted, the pile does not get the full benefit of the blow, and consequently the test is not accurate.

(2) See that the ram or monkey is raised during the test blows to the full height marked on the piling frame, and see that the men holding the guy ropes do not regulate the fall of the ram so that it loses its force just before striking the head of the pile. The precaution should be taken of checking the length of the gauge rule used for measuring the fall of the ram with an ordinary rule. It is as well to measure the gauge rod every time before it is used.

(3) Watch closely the angle at which the pencil is held in marking the lines on the pile, as a somewhat cute device which needs to be guarded against has come frequently under our observation. The pencil was held at an acute angle from the bottom of the swing gauge, thus marking the pile $\frac{1}{8}$ in. or even $\frac{1}{4}$ in. above what it would be if the pencil were held at right angles to it; then, after driving, the second line was drawn with the pencil at right angles to the gauge. Obviously, the space between the two lines was less than it would be if both of the lines had been drawn with the pencil at right angles to the bottom of the gauge. An effective cure is for the clerk of works to insist on marking the pile himself.

(4) When pile driving is carried out at night, see that the pile shoes are not knocked off before lowering the pile into position for driving.

Reinforced concrete piles are generally placed in groups of two, four, or six. It may happen that in driving the first pile of the group a set cannot be obtained, and the pile is driven right into the ground, even then failing to pass the test, always going, say, $\frac{1}{16}$ in. to $\frac{1}{2}$ in. more than is permissible. Where such an event is likely, the driving of the first pile should be stopped, and the second one started. The next day, the first pile should be given about 120 blows with a 3-ft. drop of the ram before applying the test, when, in the majority of cases, the pile will not go more than $\frac{1}{16}$ in. and probably not as much as that. Should, however, the pile still continue to go, it must be spliced or added to in some other way. The splice must be designed by the engineer; as a rule, extra rods are placed at the joint of the old pile and the addition. The concrete of the pile must be cut away down for some distance so that all the reinforcement is bared. The new steel for the additional length is properly framed up and joined to the projecting reinforcement of the pile. A wooden mould is placed round the steel framework to the height required for the additional length, and the whole is now ready to be filled with concrete exactly as for a column (described later). When the concrete is sufficiently dry, the mould is taken off, and five or six weeks later the driving may be re-started.

Cast-in-place Piles.—Concrete piles cast in place are now frequently used in foundation work, one of the best known of the type being the "Simplex" (Fig. 173), in which system a hollow cylinder is driven to a bearing, such as ballast, and then gradually withdrawn, the hole left by the cylinder being filled in with concrete and heavily rammed. This cylinder is made of about $\frac{7}{8}$-in. lap-welded steel about 14 in. to 16 in. in diameter and from 30 ft. to 40 ft. long, made up in two or three sections cross-welded. The upper portion is strengthened by a $\frac{7}{8}$-in. riveted band about 18 in. deep and a narrower band $\frac{1}{2}$ in. thick riveted to the bottom end just above the shoe. The pile is provided with a special alligator jaw or shoe, which is closed during the driving to prevent the entrance of the surrounding materials into the cylinder or pipe. The jaw, which is securely attached to the pipe by cable hinges, and can be used over and over again, is composed of two symmetrical wedge parts, which are kept closed during the driving by the earth pressure. In hard soils a detachable cast-iron point is often used, it being left in permanently. The cylinder or form is driven by a pile driver, which is very little different from the ordinary type, except that it is fitted with a strong pulling device attached to the leaders for the purpose of withdrawing the form after it has been driven. When the cylinder has been driven to refusal, the driving head is hooked to the hammer and wire attachment, and both are hauled to the top of the leaders during the concreting. A batch of concrete of about 5 cub. ft. is dropped into the form, after which the entire form is raised about 1 ft. by a pulling device. The concrete is then rammed by dropping a heavy rammer into the mass. The impact opens the jaws and forces the concrete out into the space made by withdrawing the form. The process is continued until the whole form can be withdrawn, and the space it occupied is filled with concrete, which sets and hardens, and so becomes a concrete pile. This pile is sometimes reinforced by bars being inserted and so arranged as to allow the rammer for the concrete to be dropped down. By the

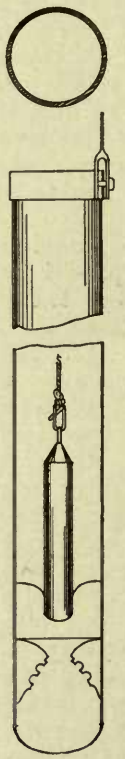


Fig. 173.—Simplex Pile

use of a wet and sloppy concrete the rammer need not be used, as the concrete will settle down and surround every part of the reinforcement. The rods inserted should project through the head of the pile for the purpose of joining up the beams at their junction at the pile head.

"COMPRESSOL" CONCRETE PILES

The "Compressol" system is the invention of a French engineer, S. L. Dulac, and its name indicates the compressing of the soil. The plant includes three rams which are of different shapes and sizes, as shown in Fig. 174. The "borer" is of a long conical

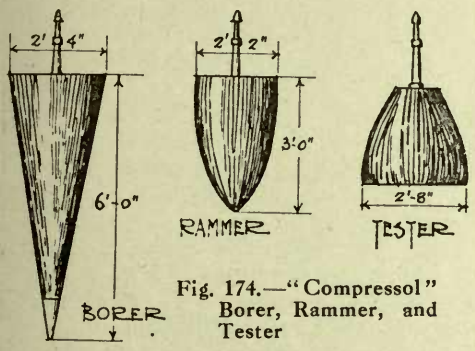


Fig. 174.—"Compressol" Borer, Rammer, and Tester

shape, with a sharp point, the latter having a special cavity which brings back a sample of the soil reached at each blow, so enabling an opinion to be formed as to the necessity of carrying the foundation to a greater depth. The diameter of this ram at the upper part is 2 ft. 4 in. and the length about 6 ft., whilst the weight is 32 cwt. The second ram is called the "rammer," which has an ogival shape with a length of 3 ft. and a diameter of 2 ft. 2 in., the weight being 30 cwt. The third ram is the "tester," and weighs exactly 1 ton; unlike the others, it is made to fall with the large base downwards. It is shaped somewhat like a frustrum, and the greater diameter is 2 ft. 8 in. These rams are worked with a frame, in a similar manner to that employed for dropping the monkey in piling work, with a steam winch and boiler; the frame is, in addition, supported on a revolving base plate which, in turn, rests on a set of wheels in order to allow the frame to be moved longitudinally, as well as about its own centre. There are practically two systems, one of which is used for deep foundations and the other for merely a surface compression of the soil when the latter is of a poor nature.

In forming the deep foundations the "borer" is first used, and this is drawn up the frame and allowed to drop on to the soil, the height varying according to the circumstances, sometimes reaching as much as 35 ft. Fig. 175 shows the frame and the "borer" in position during the execution of the work. An automatic self-acting grip, supported by a pulley and chain, is employed, and each ram is provided with a specially designed rod at the head, as shown in Fig. 174, which allows the automatic grip to come into action. As soon as the ram is dropped, the automatic grip is lowered to pick up the ram, and upon the latter being wound up to any desired level the grip comes into contact with a special ring, which is fixed to the frame, and this automatically releases the ram and allows it to fall; then the process is repeated. A sufficient number of drops is given to make a circular hole in the soil; and in the operation all the strata adjoining are compressed to such an extent that the sides of the hole hold up very well in most cases.

The process of boring and compressing is continued until the level is reached at which the foundations are to start, and then the "borer" is withdrawn and several large stones are thrown into the circular shaft and subjected to severe blows with the "rammer," which forces them outwards and spreads the bottom of the hole until it forms a wide base, in some cases with a diameter of 6 ft. or 7 ft., thus giving a good footing for the concrete pier, which is to come above.

When the base is properly prepared the process of filling in the hole with concrete is proceeded with, and this is accomplished by depositing layers about 16 in. to 20 in. thick, which are heavily rammed with a few blows of the rammer; cement concrete is used where great strength is required. The effect of the ramming is to consolidate the concrete and give it increased strength; at the same time the surrounding soil is much compressed by the spreading out of the concrete, which, when completed, has been found to measure 6 ft. and more in diameter, as against the original size of the hole, which was about 3 ft.

Fig. 176 shows the finished concrete pier, and indicates clearly that the object of the ramming is admirably obtained. The photograph shows how the concrete is forced out to a greater extent when passing through a

soft stratum, and thereby effecting a greater compression at this point, making a more uniform resistance at each portion of the pier and giving projections which materially assist in resisting any downward pressure.

It is usual, when constructing these piers in actual building work, to use large stones at the base of the pier and smaller material when nearing the top.

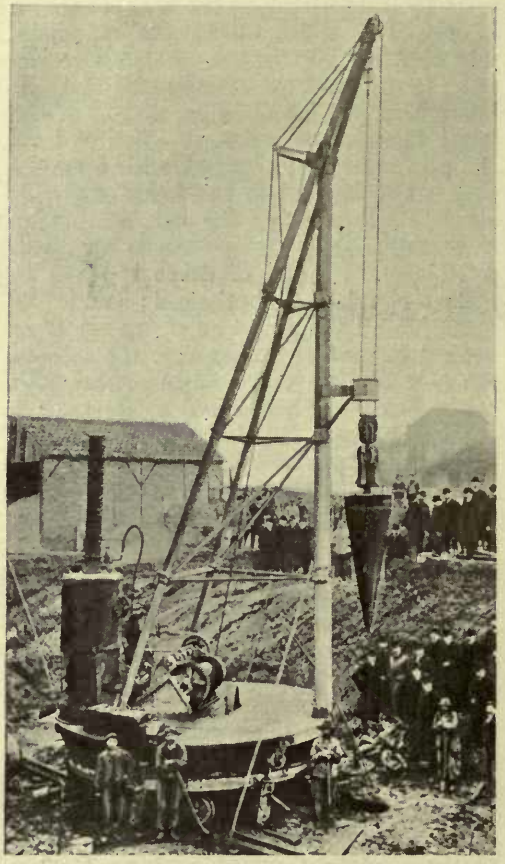


Fig. 175.—"Compressol" Frame and Borer in Use

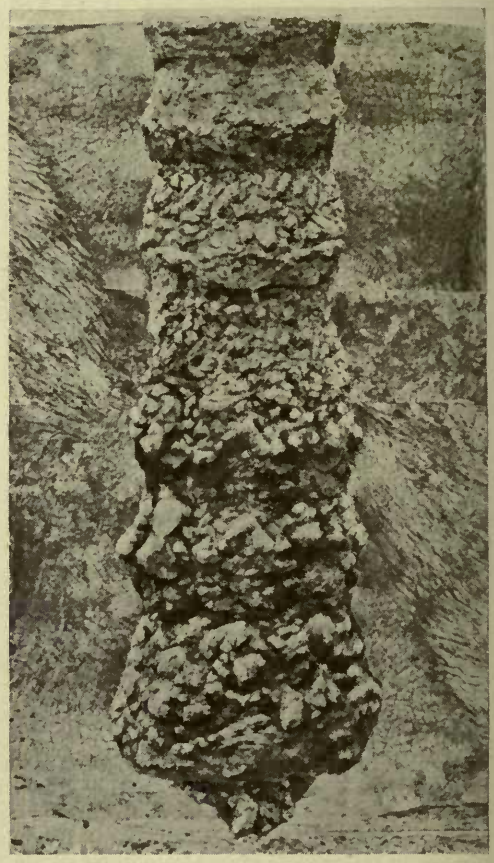


Fig. 176.—"Compressol" Pile

The bearing power of each pier is ascertained by means of the "tester." When the pier is nearing completion, the rammer is withdrawn and the tester is substituted, this being used to give a volley of heavy blows on the head of the pier, during which time the set is ascertained. Naturally, the less the pier is sunk by each blow the greater the resistance it will offer when carrying a superincumbent load. The weight of the ram being exactly 1 ton and the fall being, say, 30 ft., a force of 30 ton-ft. is obtained when the ram meets the pier; that is to say, 30 tons for a set of 1 ft. If the set is only ½ in. per blow the bearing power will obviously be 24 times as great, or, in other words, 720 tons. Against this, of course, must be placed a factor of safety to allow for the loss of energy due to vibration, the resistance of the air, and other causes, when testing.

When much water is encountered during the boring it is necessary to adopt measures to prevent it from draining into the well, and this is accomplished in a simple manner. The hole is filled with ordinary clay up to about 8 in. above the water level, and the boring operation started afresh. The clay is forced outwards and compressed around the well, forming a kind of watertight lining, which is usually sufficient to prevent any great inflow of water. It may become necessary in some cases to repeat the opera-

tion two or three times, but eventually a perfectly watertight well can be obtained.

In the case of surface compression only (which is used where the loads are light, but where the soil is "made" ground, or of a similar description and needs improvement) the "borer" is not employed, but trenches are made about 3 ft. deep and the rammer is used to make a hole 4 ft. or 5 ft. deep, which is then filled with dry stones, bricks, or similar material for about one-third of the depth, these being subjected to a few heavy blows with the rammer, which forces them outwards and downwards all round into the soil. This process is repeated two or three times and the tester is then used to give a few blows, after which a great improvement can be seen in the soil.

This method is employed in different portions of the trenches at such distances apart as may be necessary under the particular conditions of the case, which are governed by the nature of the soil and the loads to be carried. This is a very cheap and effective way of improving the foundations for a light structure.

Pile Caps.—When all the piles have been driven and passed, the constructor can proceed to cap them, and to form the connecting beams that will support the bottom floor. Should the site at the surface be of fairly solid substance, no bottom planking will be necessary for the pile caps and beams, or even for the floor slabs. Fig. 177 is a plan of a typical pile cap.

For the purpose of constructing a concrete cap to a pile that has been driven a mould in the form of a box is made to surround each group of piles. The reinforcement has to be placed in each of the boxes, according to the engineer's designs. The pile caps are connected together by beams which are moulded in wooden forms; the beam reinforcements are placed in the beam moulds, all the bars properly connected up at their intersections over the piles (see Fig. 178), and the concreting begun. While the concreting is being done, the spaces between the beams may be filled in to the level

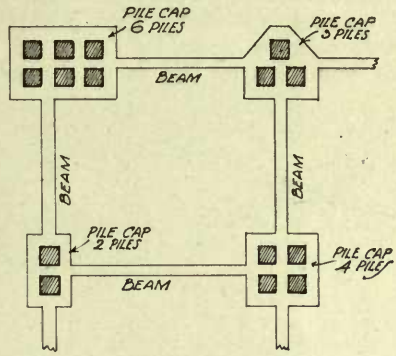


Fig. 177.—Plan of Typical Pile Caps and Connecting Beams

of the under-side of the slab, by which time it is probable that the moulds to the pile caps and beams may be struck, taken away, and cleaned for re-use. The filling in between the beams up to the level of the bottom of the floor slab should be completed by thoroughly punning and levelling off with some good, small, hard material such as a mixture of cinders and gravel, clinker, etc., so as to form a good bottom on which to lay the reinforcement for the floor slab (*see* Fig. 178).

THE FOUNDATION SLABS

The placing of the slab reinforcement is the next proceeding. The reinforcement consists of bars laid to form a meshwork, or of a steel mesh formed by welding or "weaving" fine rods, or by "expanding" steel plate. Let it be assumed that ordinary

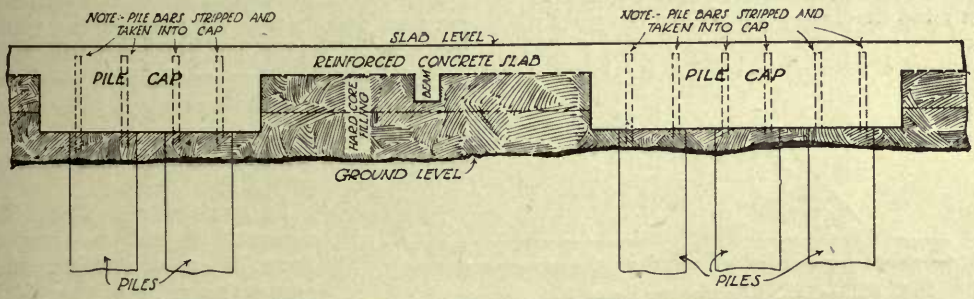


Fig. 178.—Section showing Pile Caps, Filling, Beam and Slab

round bars are to be used, spaced at certain distances, and crossed by distributing bars wired at the intersections. A practical method of placing the main bars in their correct positions is to obtain two straight pieces of boarding about 4 in. wide, and in this to cut notches for the bars to agree with the spacing specified. The boards are placed, notches upwards, one near each beam, and the bars laid to fit the notches

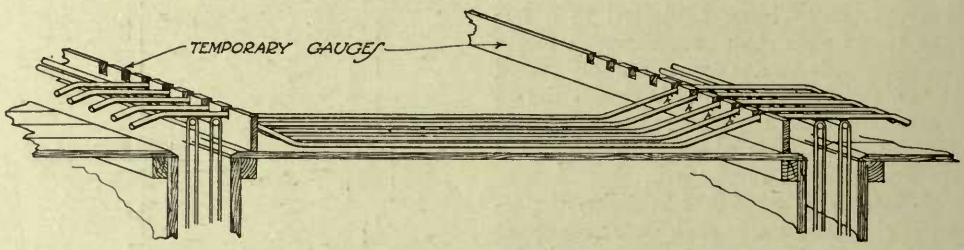


Fig. 179.—Foundation Slab Reinforcement held in Notched Templates

(*see* Fig. 179). By this means the bars will not get displaced. The notched templates can be removed shortly after the concreting has begun, quite a small quantity of concrete sufficing to hold the rods in place. However, before any concreting is done, the cross bars or distributing bars have to be laid and wired to the longitudinal bars at every (or, in most cases, every alternate) intersection. The work is not difficult or complicated, but it requires great care, and certain precautions need to be observed. For example, do not allow men to run indiscriminately over the steelwork after it has been laid down; instead, form runways with planks packed up from the ground so that they clear both the meshwork and the beams, it being borne in mind that, at this period of the work, the

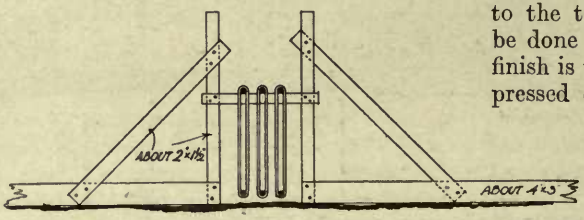


Fig. 180.—Raft Foundation Beam Reinforcement on Wooden Supports

upper or compression reinforcement and the stirrups of the beams are projecting above the level of the concrete in the beams; foremen frequently overlook this, and allow boards used as runways to rest by their ends on the projecting reinforcement, which consequently is bent anyhow.

Concreting the Slab.—Concreting the slab reinforcement already laid is the next process, and arrangements should be made, if possible, to carry out the concreting over one complete area at a time, so as to ensure the work being monolithic. There are two good practical ways of doing the work so as to ensure that the reinforcement does not come out on the surface. One is to place a round rod, about ¾ in. in diameter, under the meshwork, and roll it forward as the concreting proceeds, this method being best suited for use on a suspended floor with flat wood sheeting. The method preferred for a foundation slab, as in the present case, is to place a layer of concrete (say about 1 in. thick) from each barrow load, and to lift the meshwork through it by means of a lifting hook (any piece of steel rod hooked at the end). The hook is caught round one of the main bars, and lifted about 3 in., the mesh being given a slight shake and then lowered gently to rest on the thin concrete layer. Then the remainder of the concrete may be added, well tamping all the time, but not too heavily, or the cement will work to the top. Punning or tamping needs to be done lightly but thoroughly. The usual finish is to level the slab with a straightedge pressed down on two screeds, and worked with a short motion.

In concreting a floor, the concrete should be carried forward on a straight line across the whole of the width of a bay.

In the concreting of the slab at present being considered, those places where the columns (over the piles) and the walls (resting on beams) will come should not be smoothed over, but left quite rough, so that good joints can be made later.

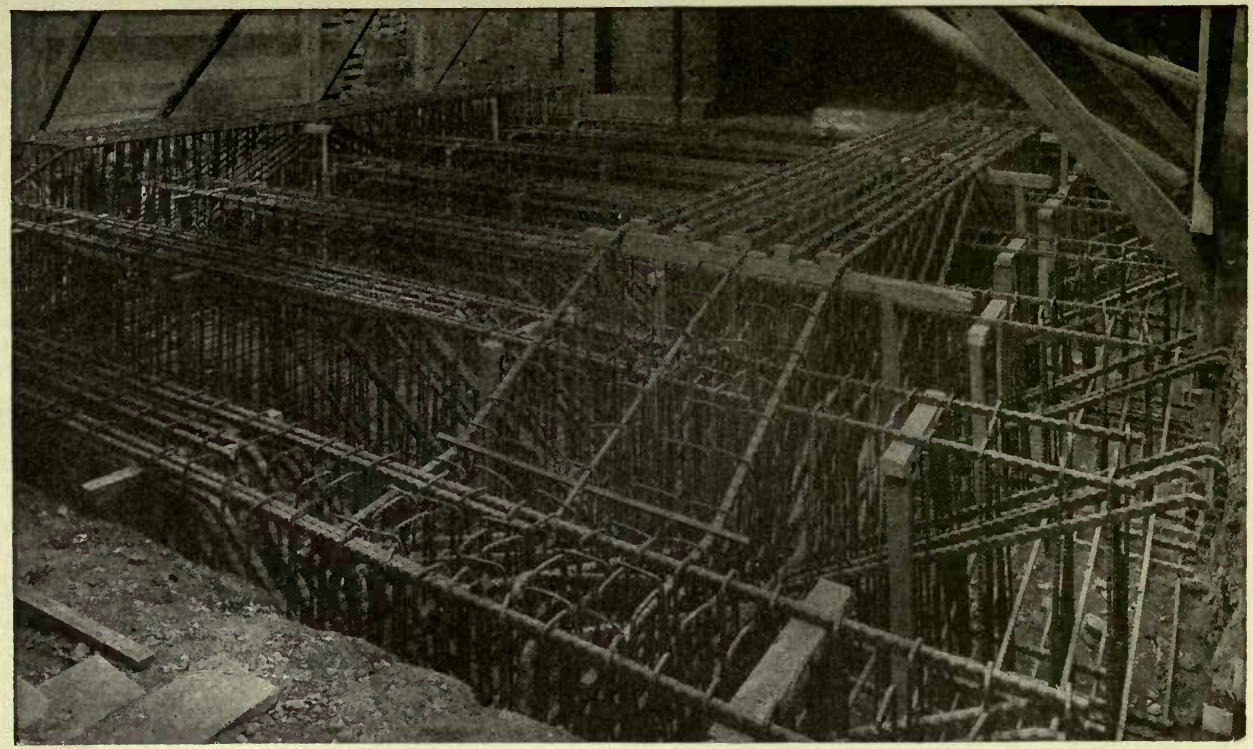


Fig. 181.—Raft Reinforcement, Kingsway Church, London

The foundation for the main building is now complete.

The raft foundation slab for the engine house is constructed in a different manner from the foundation for the main building.

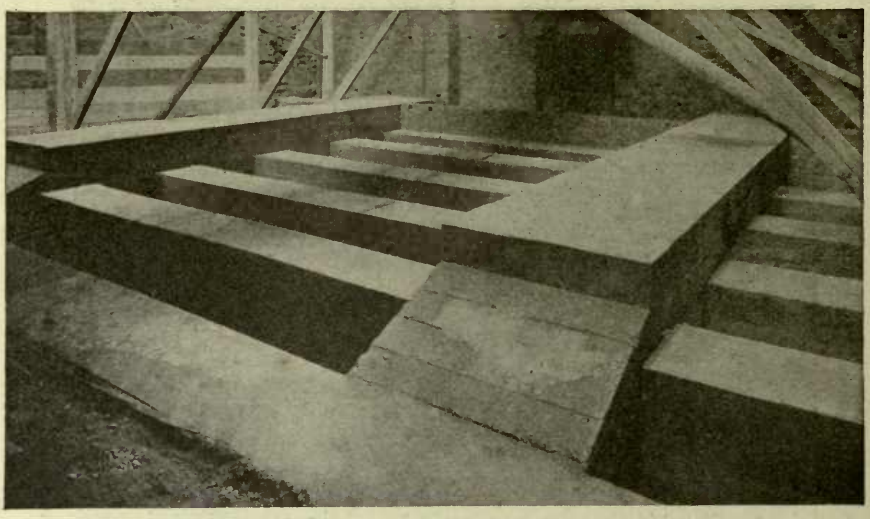


Fig. 182.—Completed Raft, Kingsway Church, London

The preliminary step is to level the site either by excavating or by making up with some good clean material, such as clinker, gravel, and ashes. The positions of the beams are next carefully set out, and the steel skeletons for the beams made in a way similar to that described on p. 133 for making a pile skeleton. The skeletons are placed in their correct positions, and set to their proper and exact levels on temporary wooden supports, as shown in Fig. 180. The slab reinforcement should then be laid in the same manner as for the floor just previously described, but in this case the concreting to the slab must be done first instead of to the beams, the concrete being taken up to the side of the beam framework, so that when it is set sufficiently to bear the weight of men and materials the wooden moulds for the beams may be erected. As soon as the concreting is set, the wooden supports to the beam framework must be withdrawn, and the framework suspended—or, rather, steadied—by braces across the top of the moulds. The moulds having been erected and fixed, the beams must be concreted, but before doing this a strong grout should be poured into the mould to strengthen the key with the slab concrete. To complete the work, the mould must be filled and the concrete carefully punned so that every particle of the beam reinforcement is properly covered. The moulds may be stuck four or five days afterwards.

Figs. 181 and 182 show the raft for Holy Trinity Church, Kingsway, London, which, on the completion of the church tower, will carry a total weight of 2,500 tons, and which is independent of the foundation of the rest of the building. A lower slab, 12 in. thick and stiffened by small cross-beams each 2 ft. wide by 2 ft. 9 in. deep, carries two main beams each 4 ft. wide by 4 ft. 3 in. deep, and the reinforcement is by means of patent indented bars.

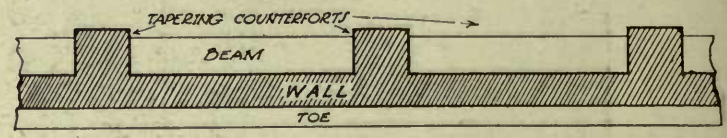


Fig. 183.—Plan of Retaining Wall with Tapering Counterforts

THE RETAINING WALL[1]

In the erection of a retaining wall supporting the roadway, the ground must be securely shored up and strutted, and provision must be made, when erecting the shuttering, to strut the ground up from the centering, but the shoring must not in the least bear on the retaining wall. For the

purpose of practical illustration, it is assumed that the vertical bars have already been placed in and secured to the foundation beam and slab, which has been specially designed for this purpose. These bars must be held upright by means of temporary stays, so that they do not become twisted.

The wall would probably be about 6 in. thick and plumb on both sides, with a horizontal beam half-way up and a capping beam at the top. There will also be counterforts which very much resemble a beam in a vertical position; they will probably be about 2 ft. 6 in. thick at the base, tapering to the width of the capping beam, which may be about 18 in.; moulds will be constructed for these similar to those for a column in a wall. The reinforcements will be carefully framed up while the concreting proceeds as the vertical rods of the counterforts will have already been fixed, as in the case of the vertical bars for the wall itself. The concreting naturally will be brought up with the wall, so that there will be no joint between the counterfort and wall. Fig. 183 shows plan of a retaining wall of this description. For a wall of this type, it is advisable to erect the centering on the external side to the whole height of the wall, and when this is done the vertical bars may be fixed to it at their proper spacing, thus obviating the use of temporary stays. The horizontal

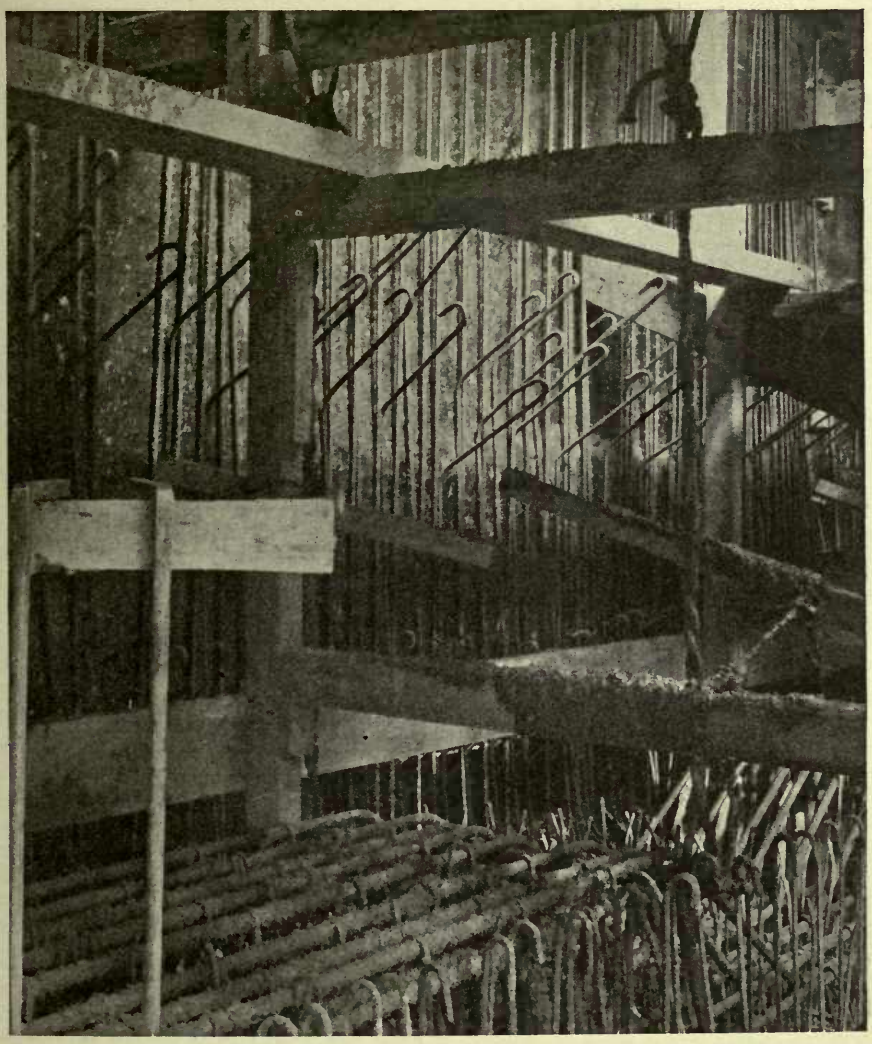


Fig. 184.—Reinforcement of Retaining Wall, Royal Insurance Building

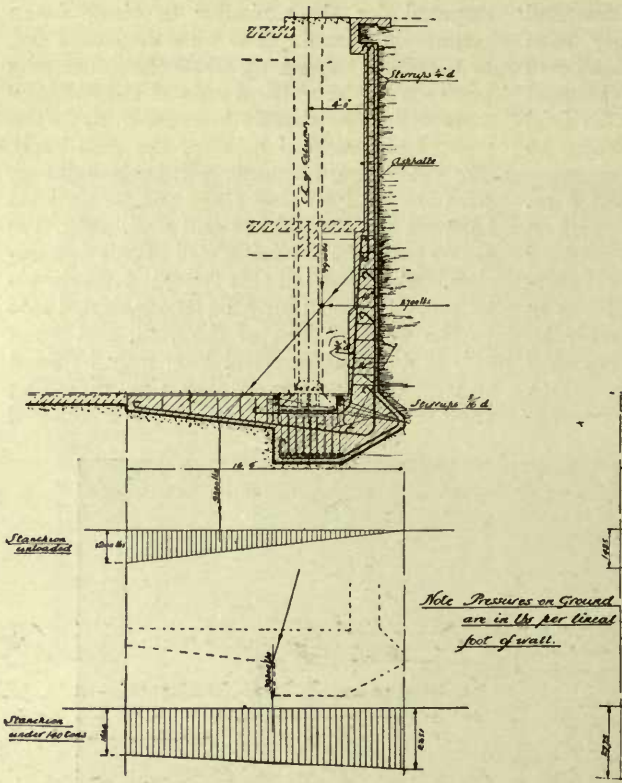


Fig. 185

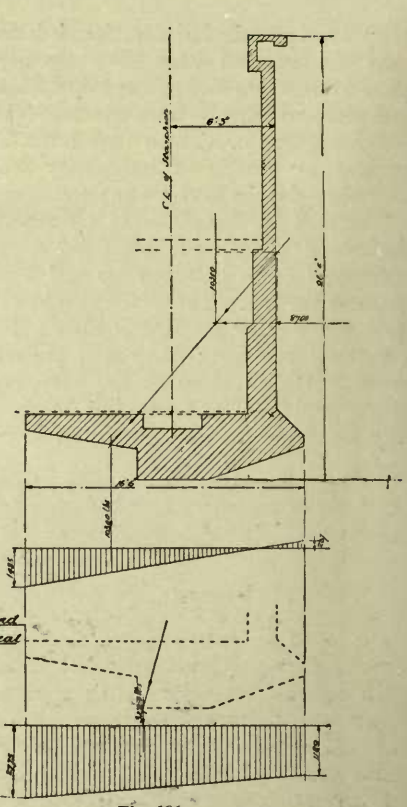


Fig. 186

Figs. 185 and 186.—Reinforced Concrete Retaining Wall for Royal Insurance Building

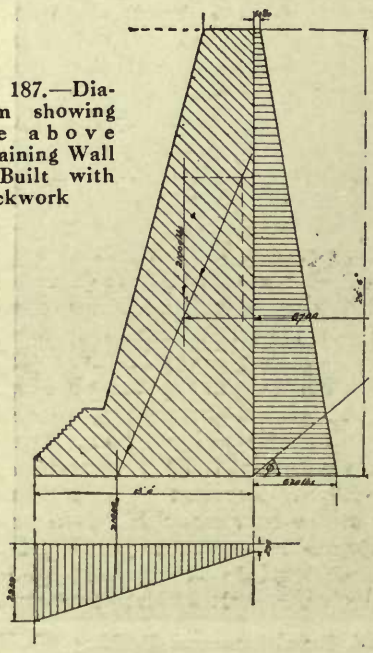


Fig. 187.—Diagram showing the above Retaining Wall if Built with Brickwork

bars should next be fixed for part of the way up and securely tied at their intersections with wire. The wall is now ready for concreting, and the shuttering—which should be about 3 ft. high—should be erected between the studs, which are generally spaced 5 ft. or 6 ft. apart. The concrete should be placed in the wall and carefully punned with a wall punning iron, doing the whole length of the wall, if possible, in one day; the next day another 3 ft. should be concreted, by which time the lower shutters may be struck, cleaned, and used for the third 3 ft., and so on to completion, the remaining horizontal bars being fixed as the concreting proceeds. When the position for the beam is reached, a two-sided mould must be securely bracketed out from the vertical studs supporting the shutters. The beam reinforcement must be placed in the mould and connected with the wall reinforcement, the concreting following.

Scaffolding will have to be erected to complete the wall, and it should be constructed in such a way that it is independent of the

wall except that it may be tied into the studs supporting the shuttering. Care must be taken not to shake the wall in erecting the scaffolding, because the concrete in the wall is still "green." (Concrete that has set but is not properly dry is in a condition known as "green.") The striking of the outside centering should not be done for some time, as the wall should be dry right through before there is any chance whatever of any weight coming on it. It is better to avoid all unnecessary risks. In a few cases where the wall is erected close against dug-out ground the centering has to remain because it cannot be got at, but this is not often so, there generally being enough room between the outside of the wall and the ground for a man to get in.

A retaining wall at the Royal Insurance Building, Lombard Street, London, is illustrated by Figs. 184 to 188, the diagrams being comparative and showing stability and pressure on ground of reinforced concrete and brick walls under identical thrusts. The space available did not allow of counterforts. The wall is 26 ft. 6 in. high from underside of toe, and is only 21 in. thick at the bottom, the thickness at the top being 10 in., there being two offsets. The wall is vertical at the back, the heel projecting for 18 in., as shown in Fig. 186. The reinforcement takes the form of vertical bars and horizontal stirrups.

COLUMNS

While the retaining wall is being built the carpenters will be constructing the column moulds, and the steel workers will be fixing up the columns. Column reinforcement may be done in at least two ways: one by erecting the bars in the mould and putting on the ties or binding as the concreting proceeds; the other by framing up the skeleton of the column on trestles as before described for

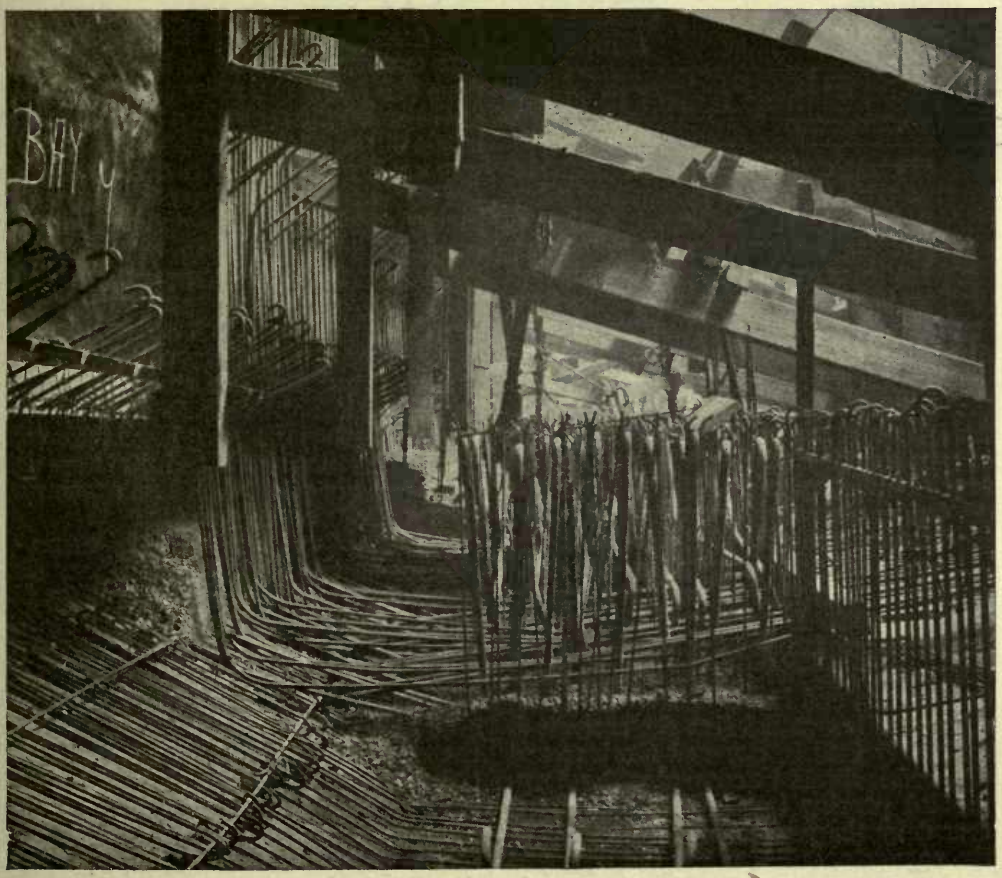


Fig. 188.—A Further View of Retaining Wall, Royal Insurance Building, in Course of Erection

piles. Probably fewer mistakes are liable to be made in the second method with regard to the exact position of the bars and the spacing of the ties, and also the ties will be more tightly fixed to the vertical bars. What is meant is clearly shown in Fig. 189 below. When the skeleton columns are made they should be carefully stacked in a place of safety where the ties will not be disturbed; this precaution is often neglected, with the result that the steelwork is trampled on and displaced as shown in Fig. 189. The skeleton columns, having been placed in the moulds, should be temporarily fixed by means of wood templates so that they stand truly plumb on all four sides; and it should be seen that the space between the outside of the vertical bars and the mould is correct. Before any concreting is permitted, the

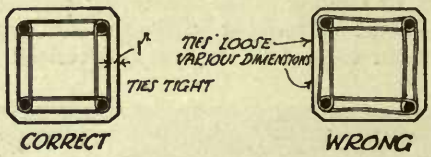


Fig. 189.—Cross Sections of Columns showing Right and Wrong Methods of Placing Reinforcements

clerk of works should inspect all columns to see that there is no dirt, sawdust, or other rubbish lying in the bottom, and that a good grouting has been poured in to form the joint between the floor and column or the joint between the column under the floor and the column above it, as the case may be. As a rule the vertical bars of a column project through the floor some 6 in. or 8 in., so that a good connection can be made with the column above it. The concreting of a column can be carried out in one of two ways—either by pouring in the concrete from the top, or concreting and punning it from the bottom, one side of the mould being left open for this purpose in the same manner as already stated for concreting the retaining wall. If pouring is the method employed, no one pouring should fill the column to a greater height than six times its diameter, and less is better. In filling columns it is essential that the concrete is sufficiently fluid to surround the bars and ties, and it should be rammed with a long-handled punning iron. Great difficulty is caused by the presence of large stones in the aggregate, and therefore the aggregate used should pass through a ½-in. square mesh.

In the construction of buildings of several stories, the diameters of the reinforced concrete columns invariably decrease as the building gets higher. The bars of one column generally project through the floors, as already stated, and they are frequently bent in slightly to meet the bars of the column above. In one system of making the joint, a sleeve piece is fitted to each bar of the lower column, and the bottom ends of the bars of the upper column enter these sleeves. By another system the bars of the upper column overlap those of the lower one for about 12 in., and the two are bound tightly together with annealed steel wire; occasionally the binding is omitted.

WALLS

While the columns are going up, the carpenters will be erecting the wall shuttering; the walls will probably not be more than 6 in. thick. The shuttering, centering, and concreting for an ordinary straight wall are worked in practically the same fashion as that described for the retaining wall, except that openings will have to be left for windows and doors, and columns (incorporated in the walls) for supporting the beams and lintels will be erected practically in the same way as described for the counterforts to the retaining wall. Extra reinforcement is generally placed round the openings; it may consist of a bar about ⅞ in. to 1 in. in diameter, bent to fit the opening with the ends, where they meet, overlapping or hooked through one another; and both the vertical and horizontal bars in the wall are bent or hooked round the bar as shown photographically in Fig. 527 (p. 324).

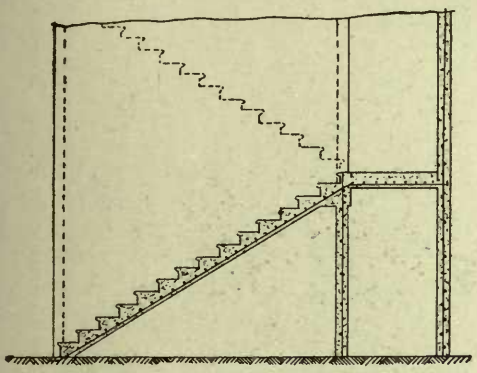
In fixing the reinforcing bars in the wall, close observation should be kept to see that the spacing or pitch of all bars agrees with the specification and drawings.

Columns, floors and walls in course of erection at the Money Order Office, Holloway, London, are illustrated by Figs. 193 and 194 (pp. 148 and 149).

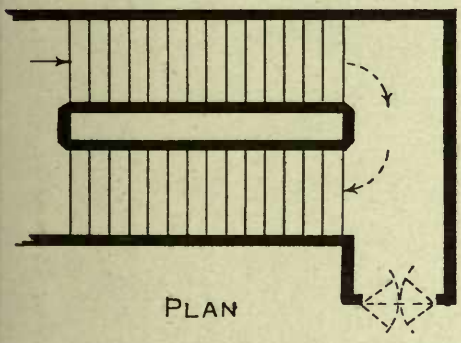
STAIRCASES

During the erection of the walls and columns provision will have to be made for the erection of the staircase, which will either be erected in situ or built up with separate treads or on reinforced concrete stringers constructed for their reception. In the present instance it is proposed to describe the former system.

The Factory Act does not allow a straight flight to contain more than fifteen treads in a single going; therefore, in most cases, when there is a height of from 12 ft. to 14 ft. between the floors, two flights must be constructed between two floors, connected by a half-space landing, with a fireproof division wall, as shown in Fig. 190. In a staircase of this type, treads are supported on a reinforced concrete carriage slab, on which they are moulded, the half-space landing being formed in the same way as



CROSS SECTION



PLAN

Figs. 190 and 191.—Horizontal and Vertical Sections through Typical Factory Staircase

the carriage. The centering for the carriage slab and landing must be erected, with moulds for beams supporting the carriage and landings. The reinforcement is then laid in position in the same way as described for a floor, and moulds are then made for forming the treads and fixed in the right position, so that the thickness (say, 4 in.) of the slab carriage can be maintained during the building of the treads. In some cases the treads are reinforced with rods laid in longitudinally and hooked into the walls. Of course, the walls surrounding the stairs must be carried up at the same time as the stairs. The concreting is begun at the lowest tread and worked upwards, being carefully punned the whole time. Stairs of this kind are generally finished in granolithic about 1 in. thick, a thinner layer invariably cracking and peeling off. The granolithic for the riser must necessarily be applied to the form or mould before the concrete is deposited; that for the treads is done afterwards, but the sooner it is done the better will be the key between the granolithic and the concrete. Treads are generally finished off by being grooved with a grooving roller, so as to prevent their becoming smooth and slippery. Figs. 190 and 191 show a typical cross section through a staircase of the type described above.

INTRODUCING THE REINFORCEMENTS INTO BEAM MOULDS

As already suggested in the case of piles, the reinforcement for beams, columns, cantilevers, etc., can be introduced in either of two ways. One is to build or frame the bars up in the moulds, and the other is to frame them up first and then to deposit them bodily in the moulds. Where members are loose and not tied to one another, the reinforcement must necessarily be built up in the moulds. Much care and patience must be exercised in this part of the work. In building a beam, the bottom bars must be kept at the proper distance from the bottom and sides of the mould, and this can be effected by using reinforced concrete blocks hollowed out as in Fig. 192 to receive the

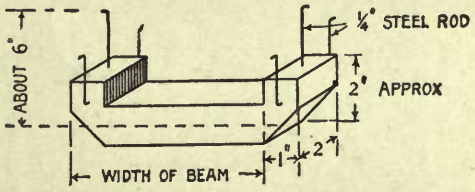


Fig. 192.—Hollowed-out Concrete Block to Facilitate Spacing of Bars

bars at the correct spacing, two bars for each beam; the blocks are laid in the bottom of the mould, and when the beam is concreted up the new concrete will adhere to the other concrete. One firm of contractors always adopts this practice, which it has found to be effective, although it might give the impression of introducing a source of weakness. Another way is to

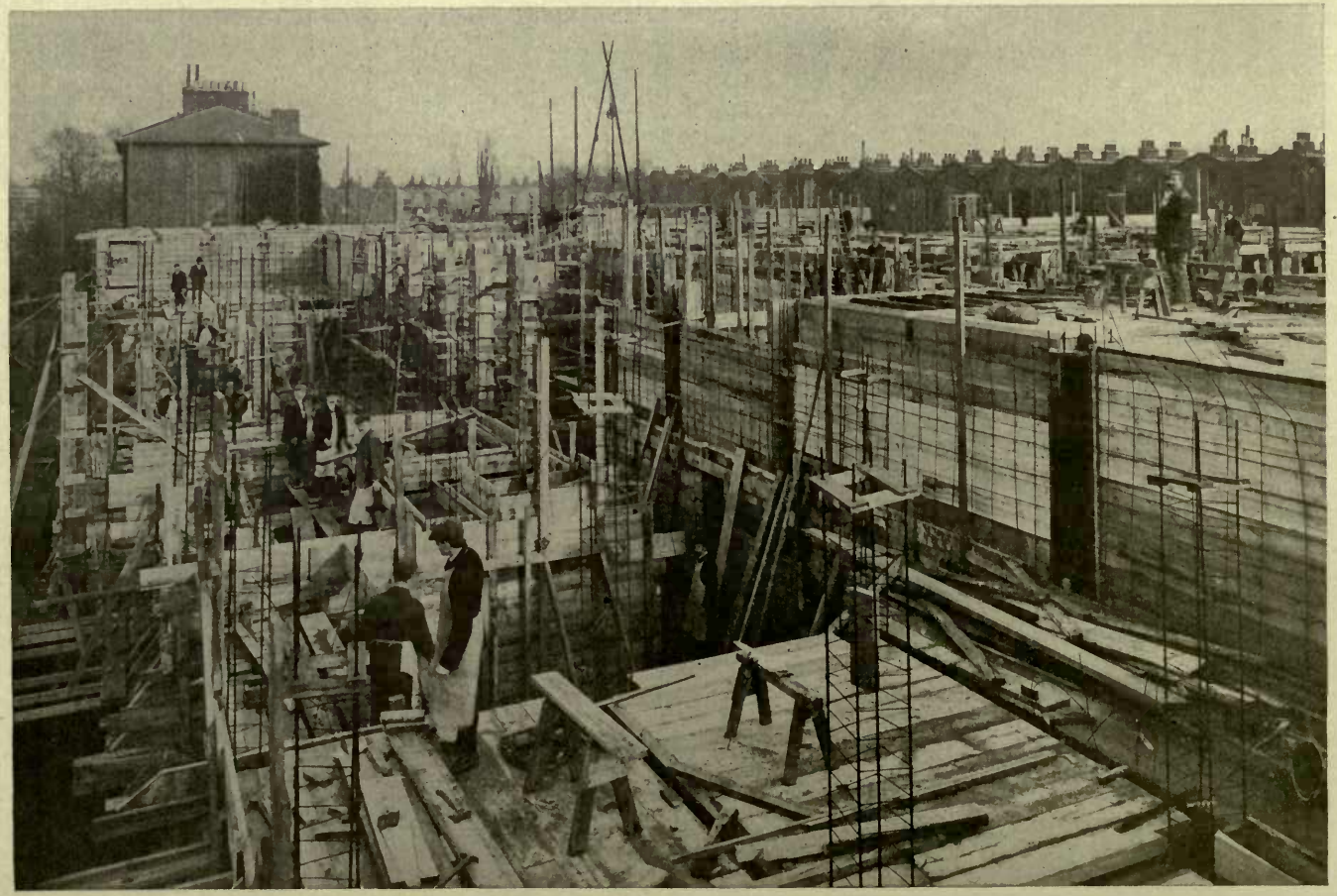


Fig. 193.—Money Order Office, Holloway, showing the Centering for Pillars, Floors and Walls

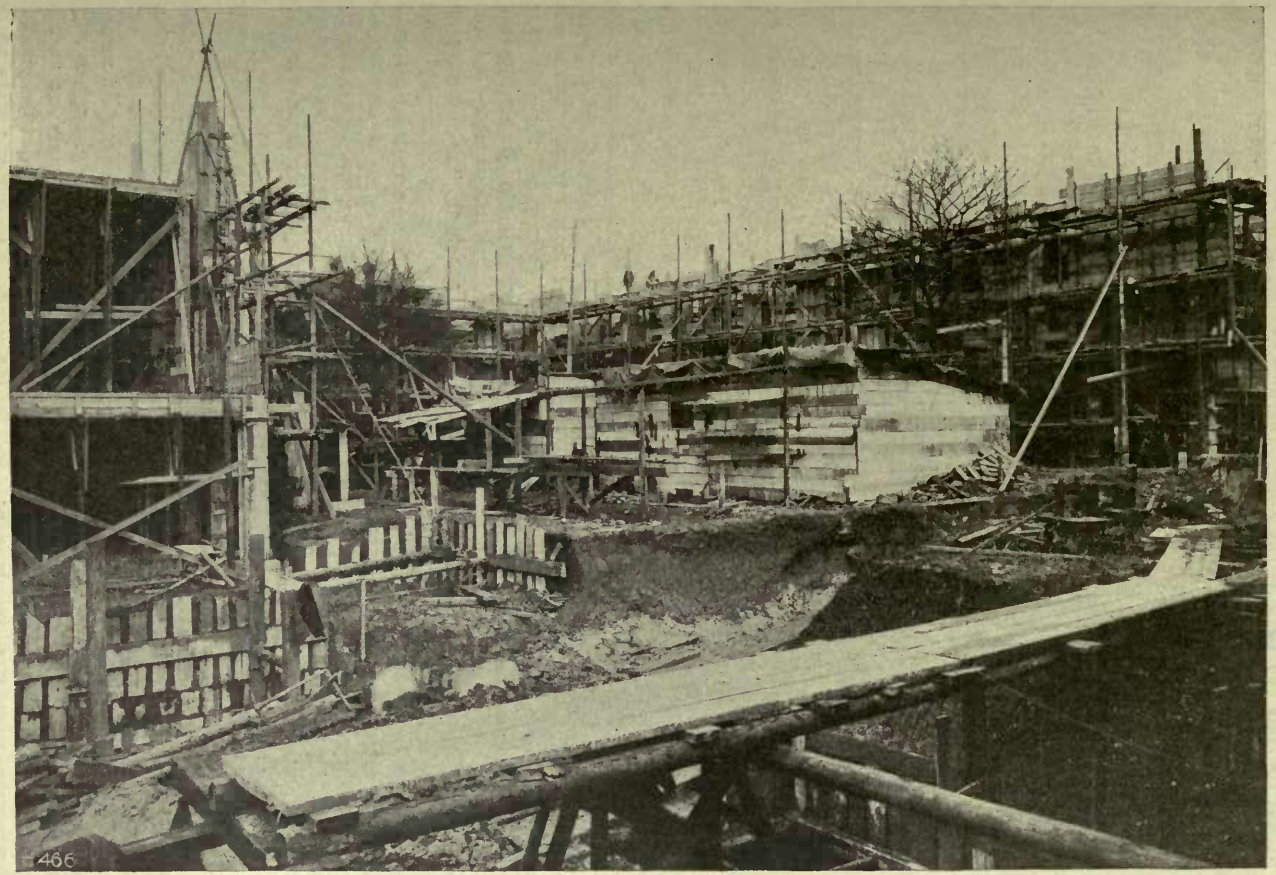


Fig. 194.—Money Order Office, Holloway, showing Two of the Wings in Course of Construction

place blocks of wood of the right thickness in the bottom of the mould, and to lay the bars on them; these, of course, have to be removed when there is sufficient concrete in the mould to hold the bars in the right position; as there is, of course, a risk of the wooden blocks being left in the mould, the foreman must watch the work very carefully.

In all cases of building a beam in a mould, the stirrups have to be placed in first, and their spacing must be correctly marked on the centering, each stirrup being adjusted after a little concrete has been deposited to hold it in place. The use of loose stirrups necessitates especial care in this particular, since it is easy for a careless workman to knock about the stirrups in all directions during the process of punning.

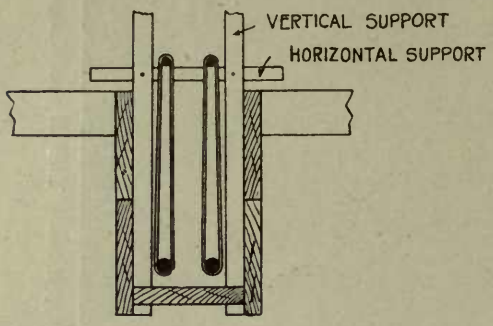


Fig. 195.—Method of Supporting Reinforcement in Beam Mould

A method of supporting reinforcements in beam moulds before concreting is shown in Fig. 195.

Difficulty is often met with at the intersection of beams over a column, there being trouble in getting the bars into their proper positions, and particularly in causing them to maintain their proper distance from the sides of the moulds. Sometimes the necessary adjustment is made—and made badly—by the aid of a sledge hammer and a considerable show of temper! The most effective method is to bend slightly one or two of the bars at their ends so enabling them to overlap properly.

In vertical work, very long bars are sometimes used, and these will always need to be properly supported temporarily to prevent their being twisted and entangled. Long straight bars can safely be bundled together and fixed to a suitable support, but when the bars have been bent to a particular form, they must be supported independently.

For the fixing of stirrups and ties of all kinds when framing up a member in its mould, their positions must always be clearly marked in pencil on the mould itself, so that, as the work is brought up, the ties can be fixed in their proper places.

FORMING SKELETON REINFORCEMENTS

For framing a member, all the bends must first be made, the bent bars being then laid on trestles, framed up in their proper positions on wooden templates, and then tied temporarily in their places while the bindings or stirrups, etc., are being fixed to them. In the case of a column bound spirally, the bars are placed in wooden templates, in which is a central hole through which passes a rod to act as a spindle; the ends of this rod are rested on the trestles, so that the whole framework can be made to revolve. (This, however, does not apply to spiral binding as designed for the Considère system, for which the binding must be carried out on a drum first and threaded on to the bars afterwards, as described for the Considère pile.) The binding is then securely fixed on a bar of the framework, and so wound round and round while the framework is made to revolve. The pitch of the binding is marked clearly on one of the rods. When a length of binding is exhausted, the end is turned tightly round a bar and another length is begun on the bar before that on which the first length finishes. The binding is continued in a like manner till the column is complete, but it needs to be securely tied on with annealed wire at every alternate bar to obviate slipping. When the binding is in the form of rings the work is done in the same manner as described for piles (*see* p. 133).

For framing up a beam, the stirrups are cut and bent and then threaded on to the beam framework, the bars of which are temporarily placed in their correct positions on templates, as in the case of a column. The spacing of the stirrups is marked on one of the bars running the whole length of the beam, and the stirrups are tied to the main bars with annealed wire accordingly. All independent members can be so framed up ready for deposition or placing in the moulds, and if not used at once they should be carefully stacked away in a place where they will not be subjected to hard knocks.

The bending of small bars round larger bars is best done with the aid of the wrench previously described.

Unfortunately it is easy for a man with a shovel, hammer, or a punning iron to cause any amount of mischief if allowed to use these tools carelessly. Extreme care should be taken in the fixing of reinforcements over points of support and also in the making of connections.

MAKING A FLOOR

The very first thing to be considered in the erection of a floor is the centering or falsework. Moulds or troughs will be needed for the reception of the main and secondary beams, and they will need to be securely strutted from the floor beneath. They must be in true alignment and of sufficient strength to bear the weight of the work that will come upon them. The wood sheeting is laid between the beams to support the weight of the floor slab and of the men working on it. Descriptions of the various methods of constructing floor centering will be found in the next chapter.

Before any steel reinforcement is placed in or on the floor centering, all the moulds for beams must be thoroughly cleared of all shavings, sawdust, dirt, or other rubbish which always accumulates during the erection of the centering.

For the purpose of preventing concrete adhering to the centering, it is advisable to close the pores of the wood by thoroughly wetting all internal surfaces just previous to depositing the concrete. When a better finish is required, the boards may be advantageously limewashed or treated with soft soap before the steel is placed in position or the concreting started; this makes the centering easier to remove and gives the work an improved appearance.

The reinforcing of a floor with ordinary round rounds was described in connection with the foundation slab. For a suspended or supported floor, the reinforcing will be carried out in the same way, but, of course, centering is holding up the floor, whereas a foundation slab rests on the ground. The reinforcement may take the form of bars and rods or of a continuous mesh made with small diameter rods or from sheet steel. But before any slab reinforcement is laid, the beams must be arranged for; the beam reinforcements must either be built up in the moulds or put together as frameworks and afterwards laid in them. Wooden blocks should be laid across the mould to support the top or compression bar of the beam; and wooden strips should be inserted on each side to maintain the correct distance between the reinforcement and the faces of the form; but great care must be taken that all these blocks and strips are removed as soon as there is sufficient concrete in the moulds to keep the bars in place. Fig. 196 shows a beam mould with the reinforcement adjusted by the means just described.

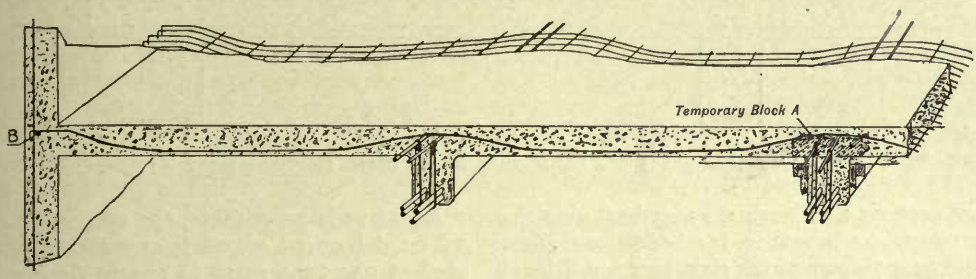


Fig. 196.—Typical Floor with Continuous Mesh Reinforcement

Concreting must not be started until all the reinforcement is complete and correct. Obviously, mistakes cannot be rectified once the concrete has been introduced.

The concreting is next carried out in exactly the same way as described for the foundation slab beams, the concrete being taken up to the level of the top of the moulds only, and left as rough and as rugged as possible, so as to make a good key for the slab concrete, which is added after the slab reinforcement has been laid.

Continuous wire mesh is generally supplied in large rolls containing from 150 ft. to 300 ft. run, and varying from 3 ft. to 6 ft. in width. For laying this to form a floor reinforcement, the rolls are hoisted on to the centering and placed side by side at one end. The roll is then undone and

the end taken into the wall (*see* B, Fig. 196) and fixed with staples to the centering, the mesh being next unrolled over the entire length; temporary blocks are placed on the beams (*see* A), over which it passes to bring up the mesh to the level required over the supports, as shown in the engineer's drawings. The correct length having been cut off, the free end of the mesh is fixed into the wall (*see* B). The other rolls are laid in the same way, side by side, the space between them being determined by the pitch of the main or tension strands of the mesh; for example, if the pitch of the mesh is 4 in. the selvedge of one roll is 4 in. from the selvedge of the other roll. Reinforcement of this type can be quickly laid, since there is no spacing and tying of rods or bars.

When sheet mesh instead of the wire mesh is used, first of all it is laid flat on the centering. The sheets may butt over the supports, for here there is an additional sheet laid near the top of the slab, these additional sheets extending over ·2 (one-fifth) of the span on each side of the centre of the support. Fig. 197 shows a cross section of this type of floor, and clearly illustrates what is meant by the above. Other mesh-reinforced floors are illustrated in a later chapter devoted to the various commercial systems.

In laying an expanded metal floor, the sheets must be so laid on the centering that the diagonal strands in the sheets all slope in one direction, otherwise the ends of the sheets will not properly key into one another where they overlap. If the sheets bulge slightly when laid in position, a few nails or staples should be driven into the centering to hold the metal down; when the centering is struck, the staples, etc., will remain in the concrete, but can easily be clipped off. As already stated, while the concreting of this type of floor is proceeding, the mesh should be lifted and shaken when there is a layer of concrete $\frac{3}{4}$ in. or 1 in. thick on which it can rest.

The concreting of suspended floors is done in the same manner as described for the foundation slab in the earlier part of this chapter; but it may be well to give in this place some further particulars of the process of concreting as applied to floor construction. The concrete should not be so thin that the water may run, and carry with it the cement, through any openings and cracks in the centering, thus causing the slab when dry to have a honeycomb structure. The more water used in mixing concrete, the greater is the shrinkage, but workmen are more likely to make concrete over-wet than over-dry because of the greater ease in handling and the less punning that will be required. The concrete should be sufficiently moist to be worked properly into the moulds and round the reinforcement.

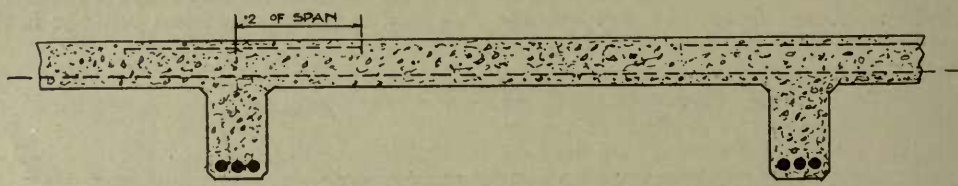


Fig. 197.—Typical Floor with Sheet Mesh Reinforcement

As far as is practicable, the concreting of a floor slab over a complete area should proceed at one time so as to ensure the work being monolithic; but when a whole floor cannot be completed in one day the work should be divided as follows:—*For a main beam*, the concreting must be carried to the centre of a column. *A secondary beam* must be filled up to, and at the same time as, the main beam to which it is connected. *In a slab*, the concrete must be carried to the centre of a beam. *A column* must always be carried to its full height. *Walls* should be worked along a complete length, and any stoppage of work should occur at a horizontal level.

Concrete spread on a floor should be carried forward on an even line across the whole width of a bay.

It will be readily understood that the remainder of the floors, columns, and walls in the factory will be carried up to the flat roof, which is exactly the same as a floor, except that there may be openings left in it for lantern lights, that the concrete, instead of being laid level, will be laid to falls, that gutters will be hollowed out to

take the water, and that a parapet will be erected in continuation of the walls.

THE ERECTION OF CENTERING

From what has already been said, it will be seen that as the work proceeds all the centering will have to be struck, cleaned, and, if possible, re-used—work in which the foreman will need all his ingenuity to effect legitimate economy, which is a different thing from scamping. The foreman must arrange that the centering is always erected in advance—well in time for the placing of the reinforcement. He should strike the centering in plenty of time for it to be properly scraped and cleaned before reframing it, but everything he does must be consistent with absolute safety. In the case of floors of large areas, the centering for the height of and including two floors should be erected as soon as possible, so that by the time half of the second floor concreting is done, the centering from the first floor can be struck and used for the next; but he must exercise extreme caution to see that the bottom planks of the beam forms and their supports are not removed until it is safe to do so. All slabs need to be temporarily supported after the removal of the sheeting. The sanction of the architect, engineer, or clerk of works should always be obtained before any striking is begun.

The ease with which centering is removed is a good criterion of the quality of its construction; but, however well it has been put together, it is necessary to employ a number of careful men to strike it. At least five or six should be detailed off for this work, and it should be definitely explained to them that the concrete is not yet thoroughly set, and that they must exercise care to prevent the jarring or chipping off of arrises, etc.

Opinions vary as to the period that should elapse before the false-work is removed; but the following is safe practice.

Floor slabs and beams.—The centering may be loosened in seven or eight days and struck in ten, except the bottom planks and supports to beams.

Wall panels.—The strutting can be removed in two or three days in moderate weather; the sooner they are struck, the better chance the wall has of drying out and hardening.

Columns, etc.—Three to five days under similar conditions to the above.

It is as well to loosen all centering before striking, in order to prevent the concrete from sticking to the boards; moreover, the air circulating through will harden the surfaces of the concrete, which will not be so liable to be chipped.

All suspended members must be left supported by temporary shores placed at judiciously chosen points, and left for anything from fifteen to thirty days. Where support is most needed is at the centres of beams and floor slabs. *On no account must any weight be allowed on any floor or against any wall for at least three weeks after the removal of the centering,* except that of the timber for the floor above and of the men working on it.

FINISHING CONCRETE SURFACES

The finishing of concrete is considered in detail on pp. 241 to 255, but it is desirable in this chapter to complete the discussion of the practical work of erecting a reinforced concrete building by giving a few notes on the subject. Concrete walls can be cleaned down, all excrescences removed, and hollows filled in, and then whitewashed—this, of course, applying only to buildings of the warehouse type. They can be decorated with plaster in the same way as any other building, but for this purpose the concrete ought to be hacked or scored to form a key for the plaster. In this hacking, great care must be exercised, because a careless man with a bush hammer or similar implement can do much damage, although this may not be visible to the eye. The hacking must be done while the concrete is green—that is to say, friable. Take the case of a reinforced concrete wall 4 in. thick that is to be plastered inside and finished in stucco outside, necessitating hacking on two sides. It is obvious that a man giving the green concrete wall hard knocks with a hammer will be causing infinite mischief by disintegrating the concrete, and thus destroying its adhesive strength and monolithic nature.

THE FIXING OF WINDOW FRAMES, DOOR FRAMES, ETC.

Provision always has to be made for the fixing of frames, etc., to the openings left for them; an effective method is to insert screws in their proper positions in the concrete, with their worms bound round with wire, and when the concrete is sufficiently

set, to give each screw a turn; it will be found that the wire is a perfect hold for the screw, and the frames, etc., can be fixed without difficulty.

The Craig screw-bore (*see* Fig. 198) is a great convenience when it is desired to attach fittings, wood finishes, etc., to the concrete. The method of using it is first to insert a master bolt, as illustrated, through a hole made in the side of the form; the wire screw-bore is then screwed on to the bolt, the reinforcement placed in position, and the ends of the screw-bore wound around the reinforcement if this is thought necessary. Before removing the forms, the bolt is inscrewed, thus leaving the screw-bore correctly placed, and, of course, most rigidly held, and forming a fixed nut into which a screw threaded the same as the master bolt is easily inserted.

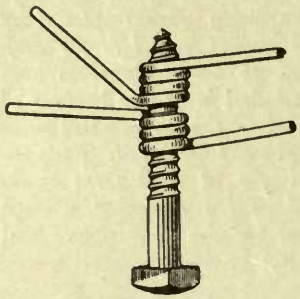


Fig. 198.—Craig Screw-bore

STEEL CORNER BARS

Steel kerbs or corner bars built into concrete steps answer three purposes: as regards appearance they give a finish to the construction, they provide a durable edge at the very place where durability is most needed, and to a great extent they act as reinforcement. United States firms who have specialised in these kerbs apparently agree that the form shown in Fig. 199 is the best for the purpose, the illustration showing the Wainwright kerb, which is made of such a shape as to be well anchored into the concrete. The bars are made in various sizes for various widths of steps. The appearance of a staircase fitted with such kerbs is shown in Fig. 200. An interesting application of the same idea is shown in Fig. 201, which is a section through a reinforced concrete column, the four edges of which are built up with corner bars made of mild steel and well galvanised. In the illustration, A indicates the corner bars and B spreaders made of stamped steel 1 in. wide and ⅛ in. thick, which maintain the corner bars in proper position while the concreting proceeds. Should additional reinforcement be required, it may take the form of upright bars, C, with wire links, D.

Fig. 199.—The Wainwright Steel Kerb

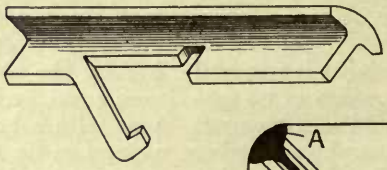


Fig. 202.—Ebco Corner Bar

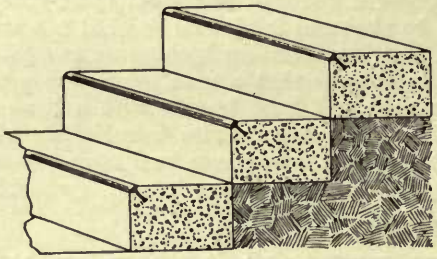


Fig. 200.—Steps with Steel Kerbs

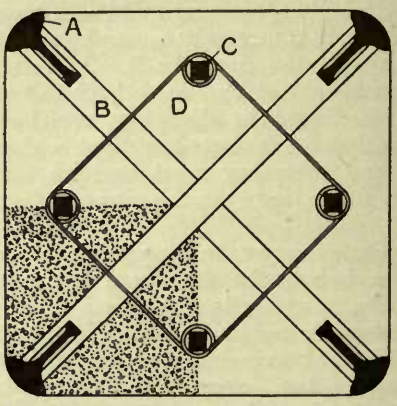


Fig. 201.—Section through Column with Steel Corner Bars

A method that is more generally employed is to insert breeze bricks at various places round the opening, so that the frames can be screwed into them.

Another type of corner bar—the Ebco—also provides a rounded corner, but is made with anchors spaced approximately 17 in. apart, as shown in Fig. 202.

Forms and Centerings

Design.—Owing to the fact that the construction of the forms, or shuttering and centering, or horsing accounts for a very large proportion of the total cost of reinforced concrete work, economy in the false-work is most desirable, and the only proper way of ensuring this is by the adoption of a correct design in the first place. This is recognised in the United States, for the scheme of the centering is usually decided in the office before the work is begun. Good design in centering is evidenced by stability and rigidity, ease of removal or striking, and facility of rapid re-erection. In the case of a building which is regular in form and contains a number of stories which are alike except as regards dimensions, thickness of floor and scantlings of beams and columns, a well-designed centering may be used many times before the roof is reached. In the case of members of unusual shape, it is only by means of proper design that excessive waste is avoided.

Centering should be designed in such a way that it may be struck readily without jarring or hacking the arrises off the concrete, and to attain this object as few nails as possible should be used, thus necessitating close supervision, for the carpenter never loses an opportunity of driving a nail. Beam and column moulds and wall panelling should be so made that the various members fit into one another without nailing, the whole being made secure by bolting, wedging, and like means.

The contractor is tempted to employ too light timber, with consequent risk to the stability, but, on the other hand, too heavy timber is unnecessarily expensive and awkward to handle. The timber used should be strong enough to take, without appreciable deflection, any load imposed upon it by the reinforced concrete, the men working on it, and the tamping. Where practicable, the scantlings of the timbers to be used will be given in this chapter.

It is essential to maintain the perfect alignment of beams, and to see that all vertical members are truly plumb, for reasons that are obvious. Further, all moulds should be carefully and sufficiently strutted so as to prevent sagging or bulging after the concrete has been deposited; and all joints should be made tight enough to prevent the thinner part of the concrete running through, but at the same time allowances must be made so that when the boards swell with the moisture they can give but not bulge.

It is advisable to add angle fillets to beam and column moulds so as to form a chamfer on the concrete, (*a*) thus obviating sharp arrises which tend to knock off when removing the centering and so reducing the amount of patching up to be done afterwards; (*b*) making spalling less easy in case of fire; and (*c*) giving a finished appearance to the work. It is a good plan to construct all beam moulds with a camber at the centre of at least ½ in.; the beam will not come out straight on the removal of the centering, but in any case, the camber looks better than a slight bulge in the middle, and it is well known that ancient Greek architects cambered all horizontal lines to make them look straight to the eye.

Failures may often be quite as much attributable to weak false-work or to the removal of the false-work before the concrete is sufficiently set, as to such a cause as bad design. Since any settlement that may take place when the concrete is newly made may easily prove disastrous, rigidity of the forms becomes essential.

The forms and centerings described in this chapter are taken from actual examples in practice, this course being considered preferable to the discussion of generalities.

SIZES OF TIMBER

Beams.—Bottom plank, 2 in. to 2½ in. thick. Sides or cheeks, 1¼ in. for secondary beams and 1½ in. to 2 in. for main beams.

Columns or pillars.—Sides, 1½ in. to 2 in.; corner studs, 4 in. by 3 in. and 5 in. by 2 in.

Slab.—1-in. boarding, maximum spacing for 1-in. boarding, 2 ft.; maximum spacing for 1½-in. boarding, 4 ft.; maximum spacing for 2-in. boarding, 5 ft.

Slab beams.—6 in. by 3 in., spaced 2 ft. to 2 ft. 6 in. apart, according to nature of slab.

Walls.—Runners, 9 in. by 3 in.

Mr. Sandford E. Thompson gives the following safe loads for timber struts in forms for floor construction :—

Strut Length	3″ × 4″	4″ × 4″	6″ × 6″	8″ × 8″
	lb.	lb.	lb.	lb.
14 ft.	500	700	900	1100
12 ft.	600	800	1000	1200
10 ft.	700	900	1100	1200
8 ft.	850	1050	1200	1200
6 ft.	1000	1200	1200	1200

The following tables have been carefully worked out and will serve as a useful guide :

Table Showing Distances Between Supports to Boards, &c., for False-work

Boards, Planks, &c.	1″	1¼″	1½″	1¾″	2″	2¼″	2½″	3″
Distances	2′ 0″	2′ 6″	3′ 0″	3′ 6″	4′ 0″	4′ 6″	5′ 0″	5′0″–6′0″

Table Showing Distances Apart for Various Scantlings in Strutting and Shoring to False-work

Scantling (*varies with height*)	4″ × 2″	4½″ × 3″	6″ × 2″	7″ × 2″	8″ × 2½″	9″ × 3″	10″ × 5″	12″ × 6″
Distances	2′ 0″	3′0″–3′6″	2′6″–3′0″	3′0″–3′6″	4′ 0″	4′6″–5′0″	5′6″–6′0″	7′0″–8′0″

Table Showing Distances Apart for Die Square Timber for False-work

Scantling	5″ × 5″	6″ × 6″	7″ × 7″	8″ × 8″	9″ × 9″
Distances	4′ 0″	4′6″–5′0″	5′6″–6′0″	6′0″–8′0″	8′0″–10′0″

ORDER OF STRIKING THE FORMS

The order of striking forms is thus summarised by Prof. Johnson : (1) Wedges in column form struck and placed in sacks for hoisting to next floor ; (2) clearance pieces ; (3) sides of column form under girder ; (4) keys at beam sides ; (5) sides of column form under beam ; (6) post up beam as close to girder as possible with new post ; (7) remove posts under girder bridge ; (8) remove bridges ; (9) place new posts under girders near beams with cross heads running along girder bottoms in same position as original post ; (10) prop up slab joists temporarily as near beams as possible ; (11) draw nails holding beam sides to beam bottom ; (12) lower wedges under posts supporting beams and turn cross heads lengthwise and wedge up again ; (13) remove beam sides ; (14) remove girder sides ; (15) remove slab sheeting and joists.

SETTING OUT

The correct position of the forms can result only from extremely careful setting out on a proper system allowing of the main lines being referred to during the progress of the work. "Horses" are made with 3-in. by 2-in. stakes driven into the ground, with floor boards, from 6 in. to 9 in. wide, and any convenient length up to a few feet, nailed to their tops, as shown in Figs. 203 and 204 ; and on these horses the significant lines are accurately marked with a knife. For a building with square angles (*see* Fig. 203), one main line having been determined and marked on horses, a line is set out at right angles, by means of a builder's square, on another row of horses, and the operation is repeated until the necessary rectangle is formed. Carefully check the dimensions

from those figured in the drawings, and do not trust to scaled dimensions. See that opposite sides agree in length, and check the accuracy of the right angles by measuring

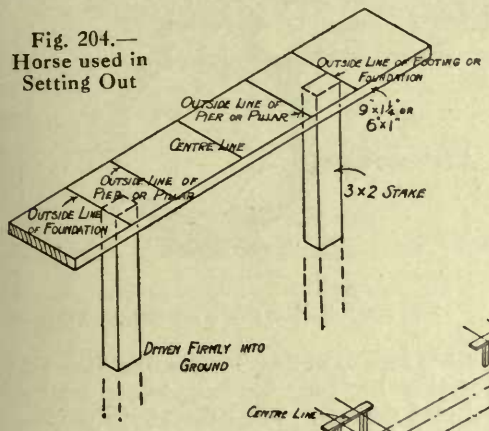


Fig. 204.—Horse used in Setting Out

30 in. one way and 40 in. the other way, the diagonal between the two points then measuring 50 in., which equals the square root of the square of 30 plus that of 40. The outside lines of the walls and columns, the centre lines, the lines of foundations, etc., may now be marked as in Fig. 203. Acute and obtuse angles are set out as in Figs. 205 and 206; in such cases the accuracy of the diagonal dimension is of the utmost importance.

FORMS FOR PILES

For a square pile the mould required is very simple; it is constructed with three sides as in Fig. 207, the uppermost side being left open for inserting the steel skeleton and concrete. Angle fillets should be fitted. The mould must lie perfectly level, and the skeleton must be securely suspended in the correct position and be dead level, so that it may be entirely surrounded by concrete. The ends of the rods must be properly placed in the cast-iron shoe at the end of the mould.

An octagonal pile is constructed as above described, with the exception that

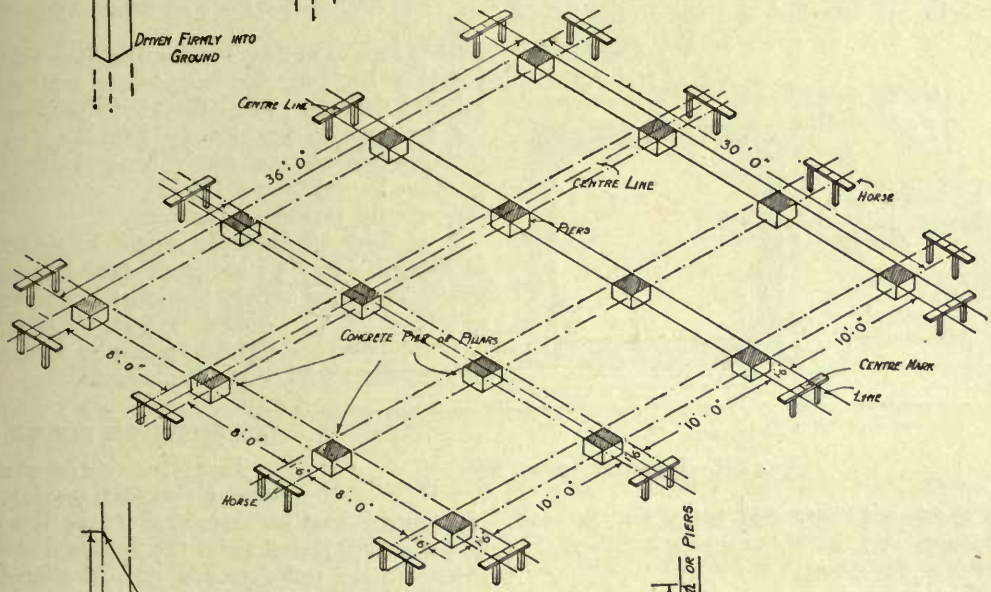


Fig. 203.—Setting out Piers of Rectangular Building

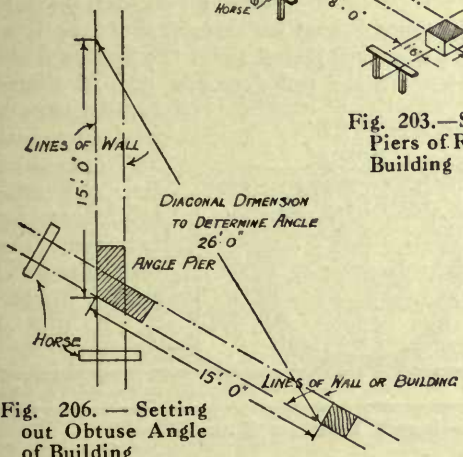


Fig. 206. — Setting out Obtuse Angle of Building

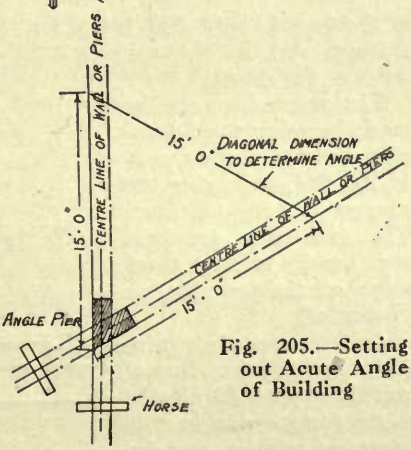


Fig. 205.—Setting out Acute Angle of Building

large fillets are fixed to the inside of the boxes.

For a round pile a mould is constructed with two circular sides or cheeks, made with narrow battens after the fashion of a barrel and bound with iron straps; the mould must give the pile two flat sides, so that they can lie flat up against the guides of the piling frame. In forming this mould (*see* Fig. 208) a straight plank is used, and through it bolts are passed about 2 ft. apart for the purpose of holding down the straps that bind the circular sides. This plank is laid down dead level on sleepers. The skeleton frame—the reinforcement—is next laid on this plank, and then the circular sides or cheeks are attached and the straps bolted at the top, as shown. The open space

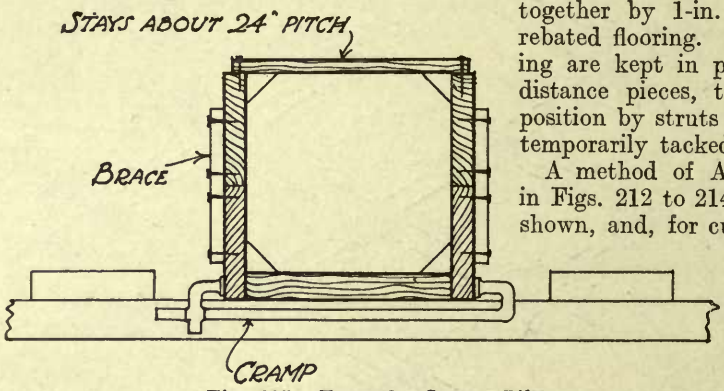


Fig. 207.—Form for Square Pile

at the top of the mould is left for the concreting, and is the same width as the bottom flat plank.

The same shape can be made by using wood templates of the shape shown in Fig. 209, and fixing the narrow battens to them. The two sides are held together by nailing short stays on the top side, as illustrated.

As in the case of a square pile, the skeleton framework is suspended by hooks to the top stays to keep it in correct position, and the hooks are removed when the concrete is half-way up the mould, so that there is no chance of the steel framework being disturbed from its correct position.

Most contractors set up a platform of joists and rebated flooring on which to make and cure a few piles. Figs. 210 and 211 show the method. The side shuttering of the

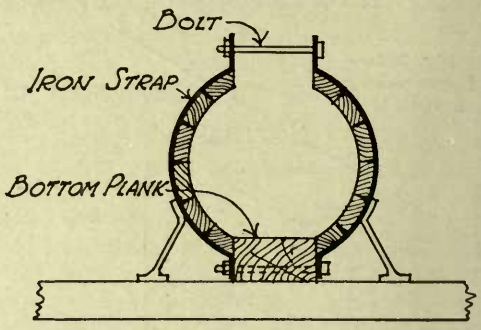


Fig. 208.—Bolted Form for Round Pile

piles is formed of 2-in. by 8-in. verticals held together by 1-in. by 7-in. cleats laid on rebated flooring. The sides of the shuttering are kept in position by 2-in. by 4-in. distance pieces, the whole being kept in position by struts and 1-in. by 7-in. boards temporarily tacked to the top.

A method of American origin is shown in Figs. 212 to 214; the parts dissociate as shown, and, for curing, the green piles are placed on 2-in. by 12-in. planks resting on beams 3 ft. 6 in. apart.

FOUNDATION FORMS

In preparing the site for a foundation slab, the ground must be brought to a true level and boarding placed about the footing if the nature of the soil demands it. If ribbed foundations are used they must be excavated for and suitable boxing provided. The concrete is poured into the

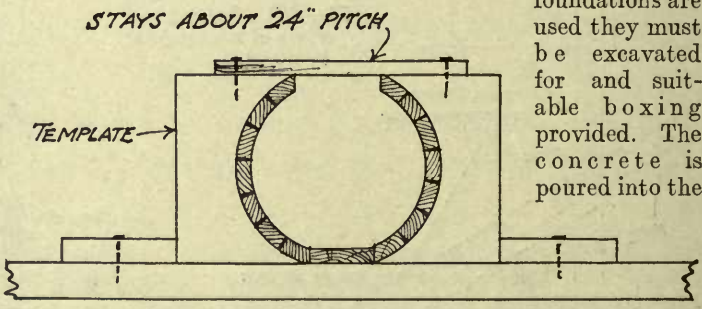


Fig. 209.—Stayed Form for Round Pile

forms in layers and the reinforcements added at the proper levels. Fig. 215 shows how the foundation forms are fixed up when the slab has had sufficient time to set hard enough to bear the weight of the men and materials.

The moulds for the beams are very readily constructed, it being only necessary to form the two sides. When these sides are fitted up in their proper positions they must be firmly stayed and strutted to each other, both top and bottom as shown. By carrying the top stays over the width of the end beams and strutting the off-side of these to long pegs driven into the ground (if it is firm enough to hold them), the mould will be secured without breaking away. If this method of strutting from pegs is impossible, the moulds may be held by bolts passing through each side of the moulds about 2 ft. 6 in. apart, as shown dotted in the illustration. They can be withdrawn when the sides of the moulds are struck. Fig. 216 is a photograph showing forms for pile caps, etc.

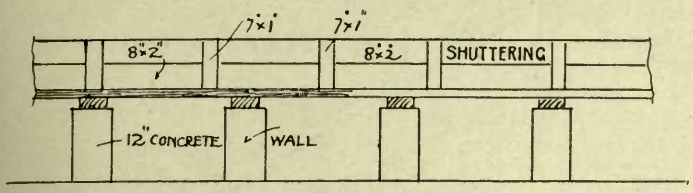


Fig. 210

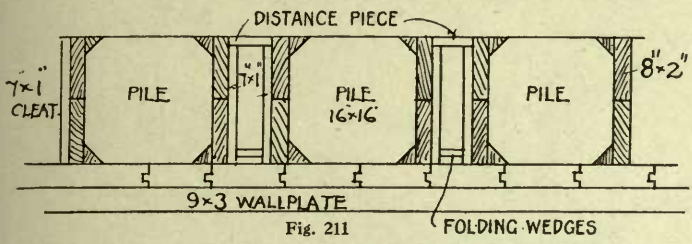


Fig. 211

Figs. 210 and 211.—Elevation and Enlarged Cross Section of Pile-making Platform

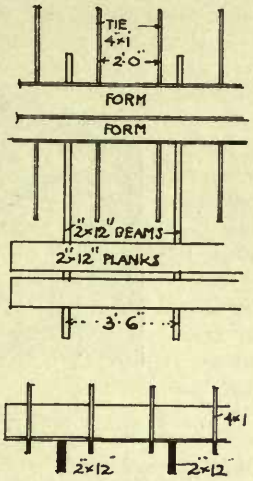


Fig. 212.—Plan and Elevation of American Pile-making Forms and Platform

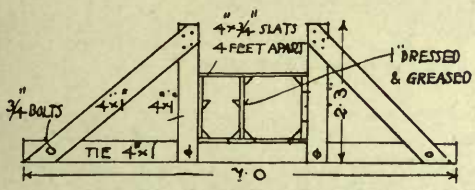


Fig. 213.—Enlarged Cross Section through Form shown in Fig. 212

Fig. 214.—Concrete Piles and Forms Dissociated

The ordinary box mould for foundations is simply four sections of the required depth, two of the sides extending beyond the others, all battened and bolted together (*see* Fig. 217). The 2-in. by 2-in. arris fillet and the 4-in. by 2-in. batten on the extended side form a groove for the battened section to slide into. The 4-in. by $1\frac{1}{2}$-in. battens are a 4-in. by 3-in. batten once sawn. The square-headed $\frac{3}{4}$-in. rods have a minus thread, and 6-in. by 6-in. by $\frac{1}{4}$-in. square plates act as washers. By alternating the fixed square

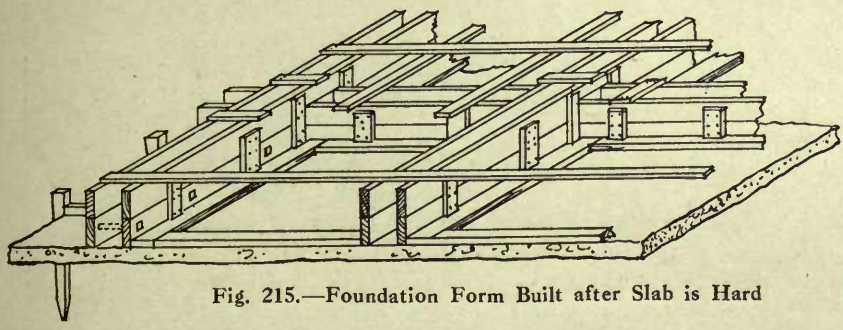


Fig. 215.—Foundation Form Built after Slab is Hard

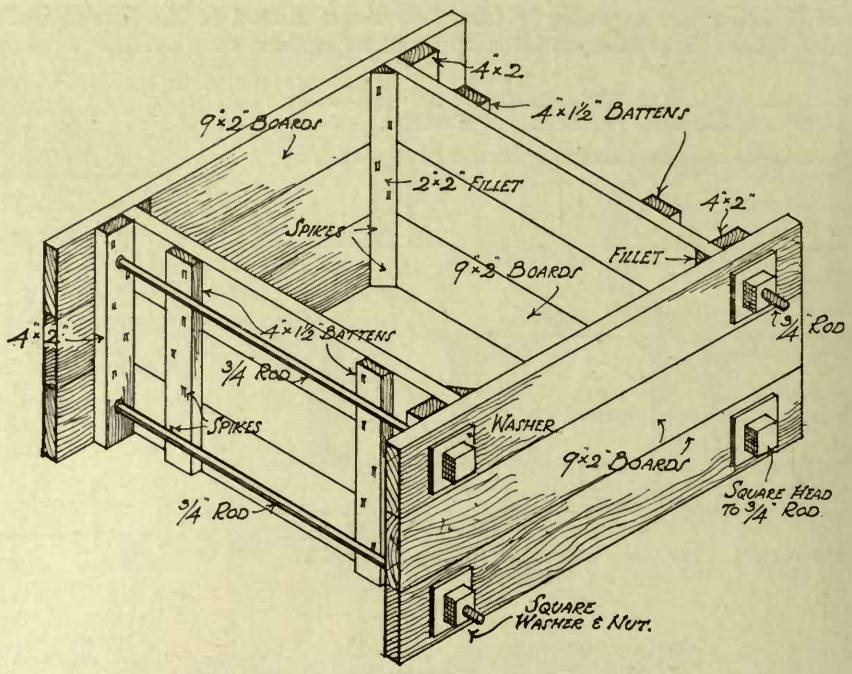


Fig. 217.—Typical Box Form for Foundation

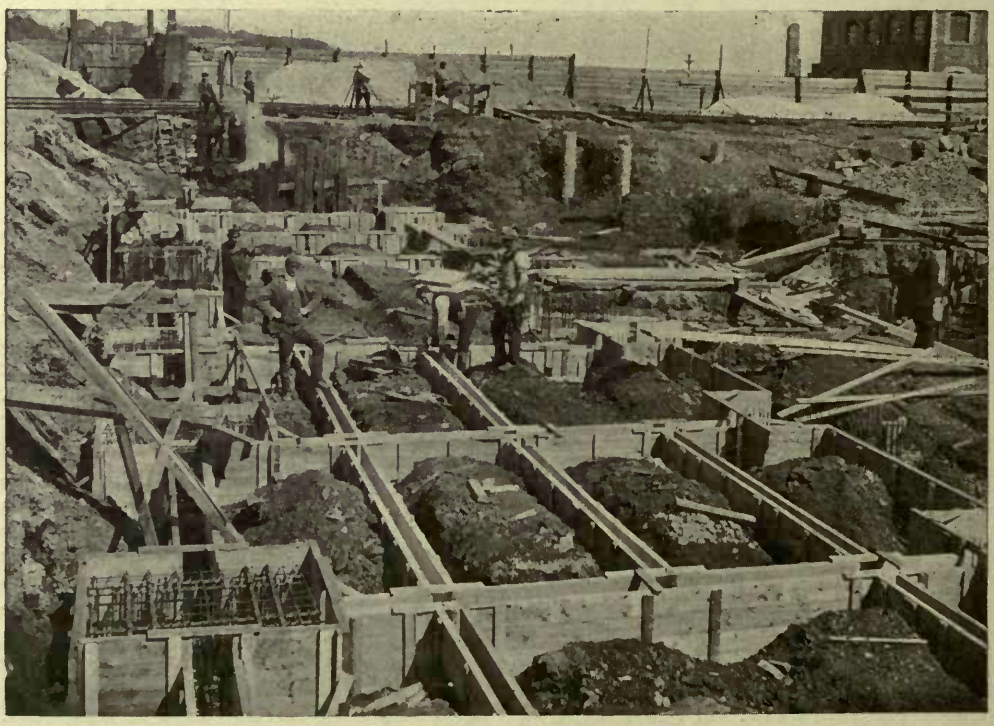


Fig. 216.—Forms for Pile Caps, Foundations, etc.

head of rod and nut the form is made capable of easy adjustment to bring it square and can be readily removed and altered.

In the type of form shown by Fig. 218, the 3-in. by 2-in. braces keep the mould rectangular and also serve as handles. This is a cheap and effective type of form which can be recommended for repetition work.

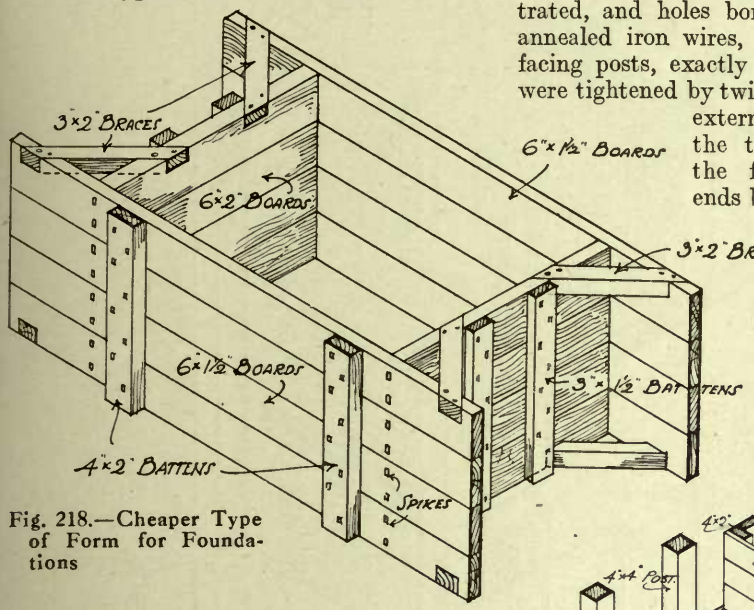


Fig. 218.—Cheaper Type of Form for Foundations

Form Strengthened by Twisted Wire.—In the United States, foundation and other forms are often held together by means of twisted annealed wire, all bolted rods and shoring being dispensed with. While English engineers might fear that die-square results could not be obtained by such a method, it must be admitted that it is cheap, simple, adaptable to a wide range of applications, and deserves to be more extensively employed. Fig. 219 shows such a form, the detail at the foot of the figure illustrating the type of foundation—square tapering piers arched together—for which the particular form shown was required. The sides consisted of 1-in. boarding nailed to 2-in. by 4-in. battens. Two opposite sides were 14 in. wider than the pier, and to the edges of their inner sides were spiked 2-in by 4-in. battens to serve as stop pieces. The four sides having been placed in position on the rough base and nailed together through the stop pieces, 4-in. by 4-in. posts were skew-nailed on the outside as illustrated, and holes bored for the passage of annealed iron wires, which connected two facing posts, exactly as shown, and which were tightened by twisting with a rod. The external wires were cut when the time came to remove the forms, the projecting ends being hammered in and finished off with a trowel. It is advisable to introduce strainers to take the stress caused by tightening up the wires, and to prevent the form being pulled out of shape.

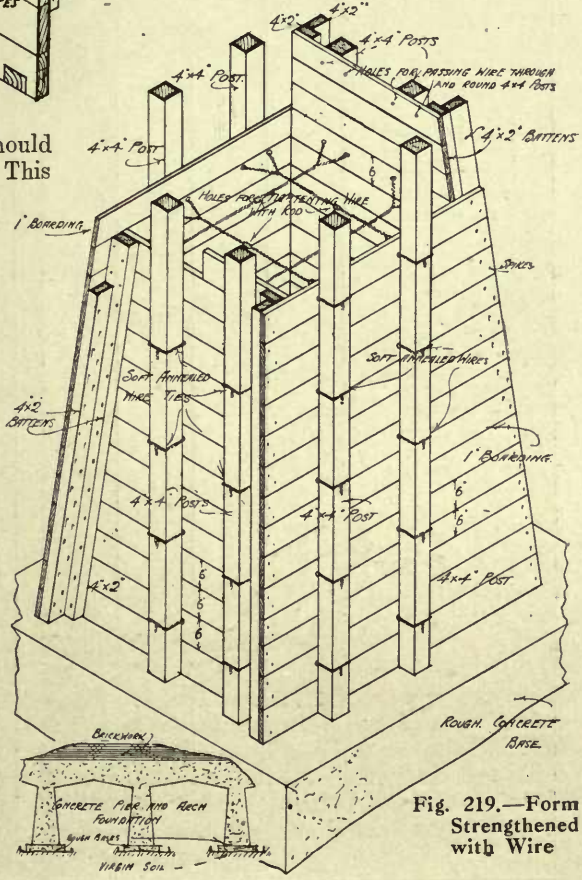


Fig. 219.—Form Strengthened with Wire

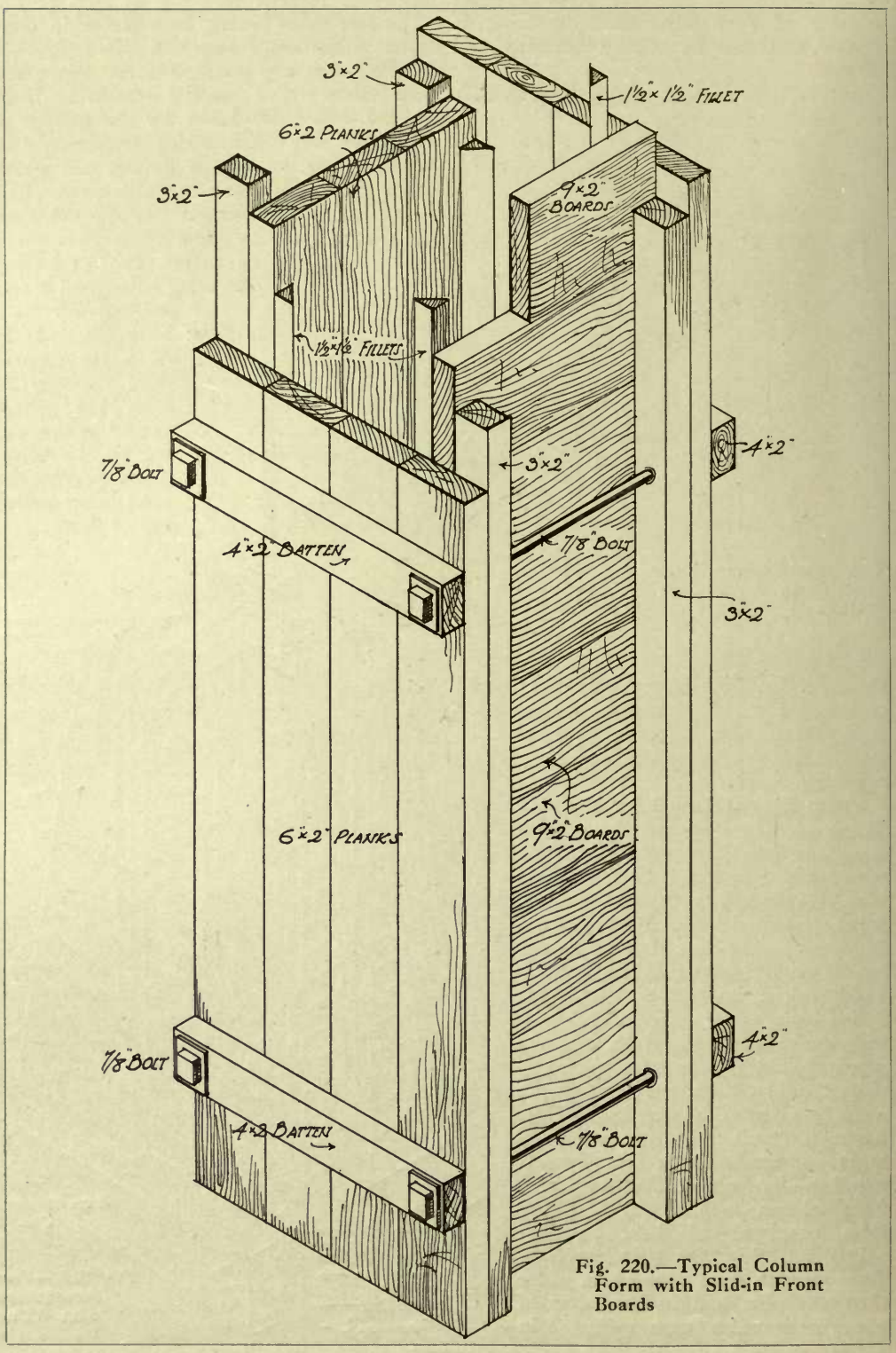


Fig. 220.—Typical Column Form with Slid-in Front Boards

COLUMN, BEAM, AND SLAB FORMS

The most frequently used forms in reinforced concrete are those for columns, beams and floor slabs, so that any economy effected in their design enabling their re-use without waste is sure to tell favourably in the cost of the structure.

Three Methods of Constructing Column Forms.—There are three methods of setting up column forms.

The first is to construct the forms for the whole four sides at once with vertical boarding held in position by battens, this method necessitating the pouring in of the concrete from the top, and consequently the use of a wet mixture, because ramming would probably displace the reinforcements.

The second method is to build up the whole four sides gradually with horizontal boards held in position by battens at the angles, and brought up in short sections as the work proceeds.

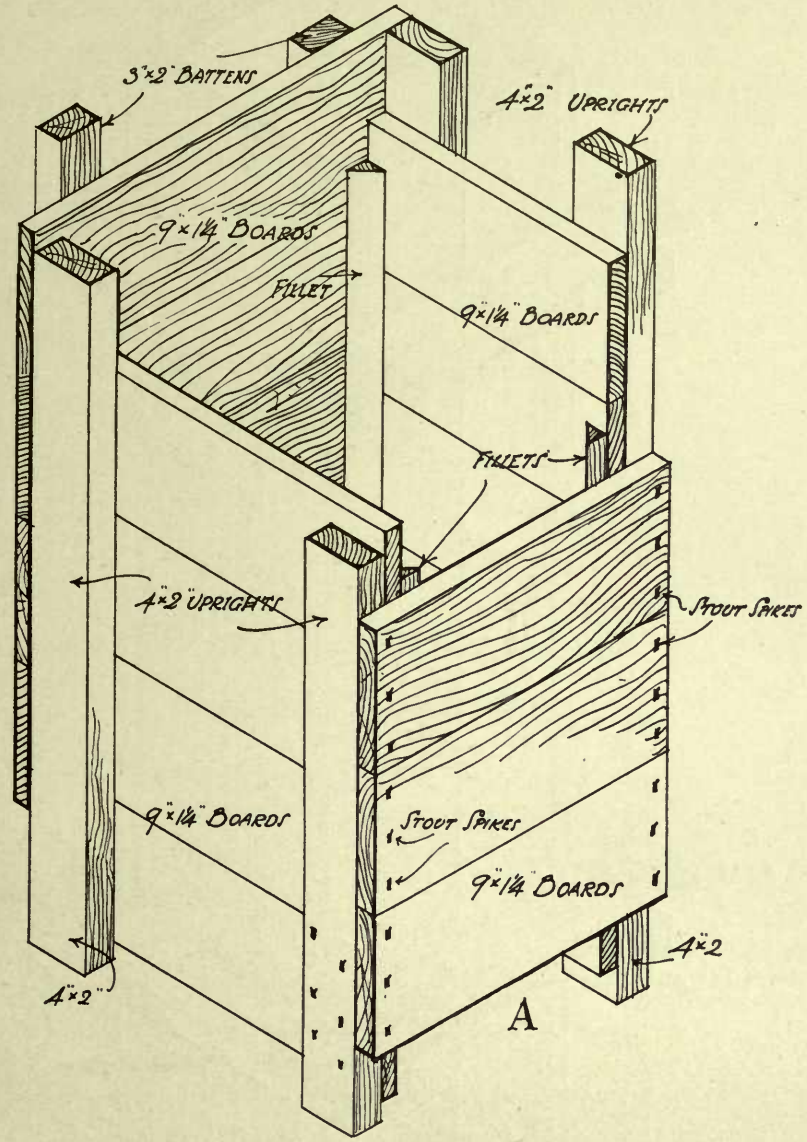


Fig. 221.—Typical Column Form with Spiked-on Front Boards

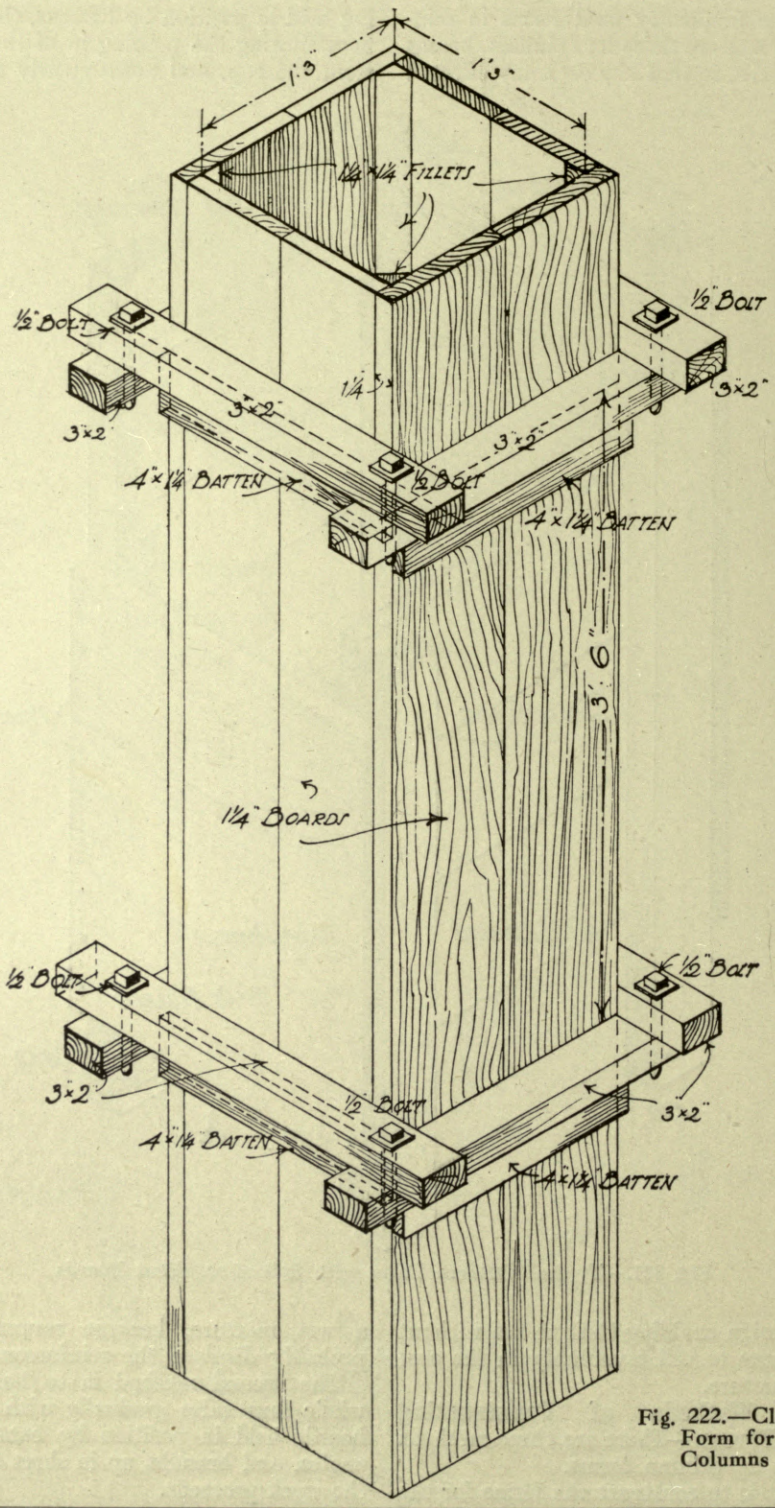


Fig. 222.—Clamped Form for Short Columns

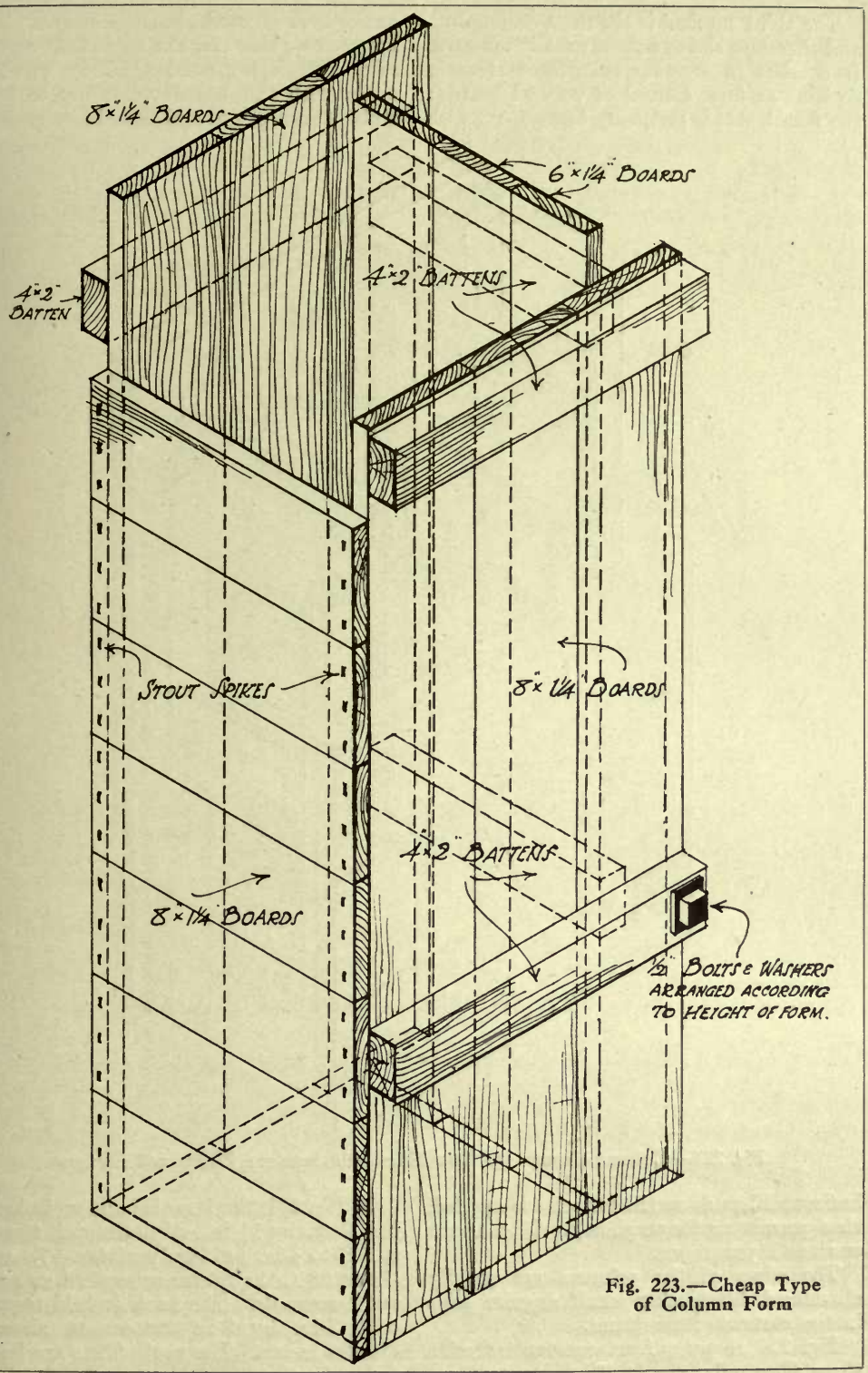


Fig. 223.—Cheap Type of Column Form

The third method is the most economical, and therefore the one most usually adopted. It consists in erecting only three sides in position at first, formed of vertical boards; the fourth side is gradually brought up with horizontal boards as the concrete is applied, thus permitting thorough supervision of the work as it progresses.

The edges of the columns are usually chamfered by placing small angular slips in the corners of the forms.

Fig. 220 shows a form constructed with three sides only, the fourth being built up as the work proceeds, in short lengths placed between a guide and the angle fillet on each side. The other portion of the mould is made with the boards vertically battened and distance-pieced and bolted into position.

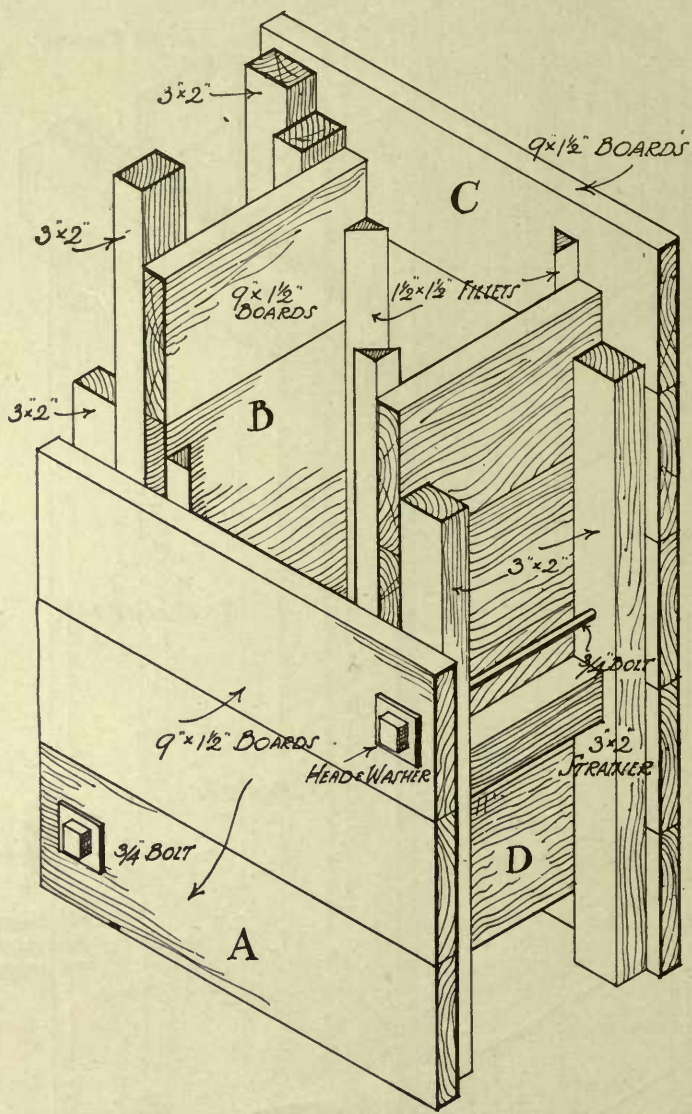


Fig. 224.—Column Form with Two Sides held between Fillets and Battens

In Fig. 221, the form is built up in boarding, 9 in. by 1¼ in., all horizontal, strongly nailed to 4-in. by 2-in. uprights. To make the front A, the boards are spiked on as the work proceeds. The back is held together with 3-in. by 2-in. battens to allow of quicker re-use. The angle fillets are lightly spiked in place.

In the type shown by Fig. 222, the boarding is vertical on all four sides and held together with 4-in. by $1\frac{1}{4}$-in. battens strongly spiked on. In addition to the ordinary spiking, wooden clamps, out of 3-in. by 2-in. stuff, and bolted together at the angles, keep the form in shape; they rest on the horizontal battens. This method of construction is employed only for the shorter height of pillars, and has the disadvantage of not allowing easy inspection or close punning, and of causing derangement of the reinforcement in the process of depositing the concrete in the mould. Spikes, or nails, clenched at the angles, are sometimes used instead of bolts.

Fig. 223 illustrates a cheap type of form in which the boards, placed on as the work proceeds, are simply spiked or strongly nailed on to the built-up three sides; 6-in. by $1\frac{1}{4}$-in. and 8-in. by $1\frac{1}{4}$-in. boards are held together with 4-in. by 2-in. battens on each of the three sides, with $\frac{1}{2}$-in. bolt rods arranged at intervals according to the height of the pillar. The life of this type of form is not long, because of the spiking on of the front boards.

In the method shown by Fig. 224, the 9-in. by $1\frac{1}{2}$-in. boards are all horizontal and are battened together on sides A, B, and C. Side D is inserted as the work rises, as short lengths of $1\frac{1}{2}$-in. board. In addition to spikes, the form is bolted through, as shown, and sometimes a strainer is inserted to take the pressure of the bolts.

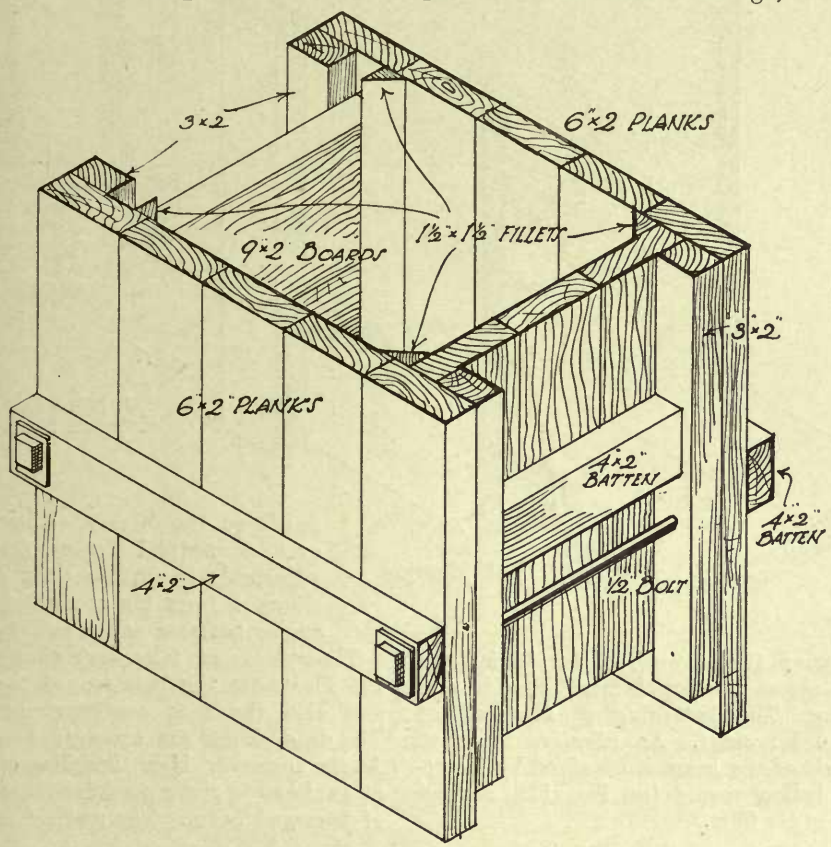


Fig 225.—Column Form with Two Sides held between Fillets and Battens

Fig. 225 shows a case in which 6-in. by 2-in. vertical planks are battened together on three sides of the form and upright members, 3 in. by 2 in., are spiked on the sides so as to make with the angle fillets two sets of grooves to receive the back and front, the latter being slipped in from the top as the work proceeds. As illustrated, the batten on the back is clear of the bolt.

The forms for beams are made in a number of ways, the best being to have two

sides and a base easily detachable (*see* Fig. 226). The 7-in. by 2-in. planks are joined together by 4-in. by 2-in. battens to form the base, while 9-in. by $1\frac{1}{2}$-in. planed boards, with 4-in. by 2-in. battens form the sides. Temporary ties, say $4\frac{1}{2}$ in. by $1\frac{1}{2}$ in., are nailed across the top before filling in and punning. The battening is arranged at 3 ft. 6 in. apart. An effective finish to the arris of the beam is obtained by inserting a hollow mould (*see* Fig. 228) instead of the angle fillet.

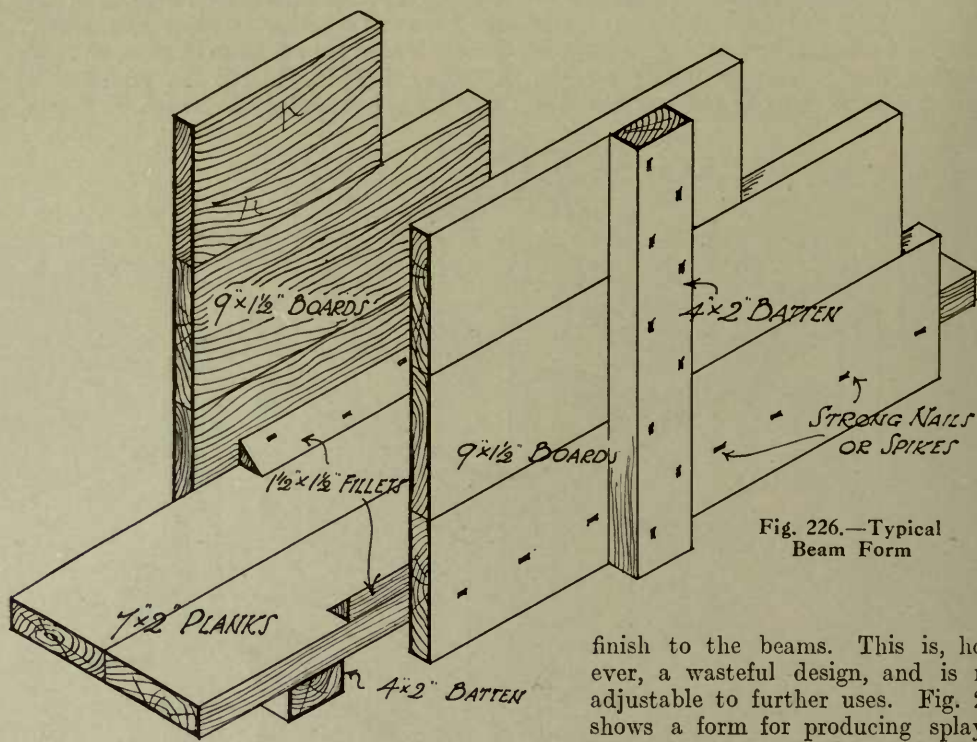


Fig. 226.—Typical Beam Form

In using dry, sound timber, a provision to allow for swelling is made by running the stuff through a mill arranged to splay one edge, as shown in Fig. 227. It is advisable not to run it to a feather edge, but to allow, say, a $\frac{1}{10}$-in. fillet.

Fig. 228 shows a beam form constructed with 2-in. planks battened together with 4-in. by 2-in. stuff, the battens to the base being extended and strutted as shown with 3-in. by 2-in. struts. A half-round mould is inserted in the angles to give a rounded finish to the beams. This is, however, a wasteful design, and is not adjustable to further uses. Fig. 231 shows a form for producing splayed angles between pillar and beam.

The system of false-work shown by Fig. 230 illustrates the junction of two beams, and also the floor centering in position. The dead shores are arranged at intervals. At the edges the $1\frac{1}{4}$-in. boarding is rounded off as shown to give a good finish to the angles of floor and beam. The method of striking the floor centering is to withdraw the 4-in. by 2-in. batten, which is put in in convenient sections; drop slightly the 9-in. by 3-in. joists, and then take out the boarding. Immediately support the floor with head and cill piece for a further period. This design is open to many objections: splayed

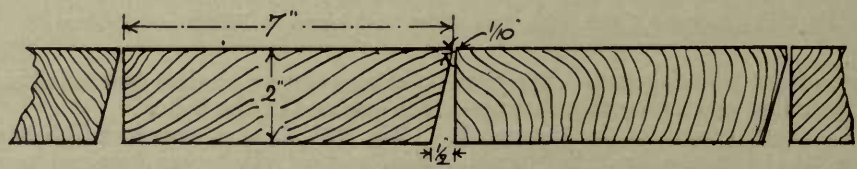


Fig. 227.—Boards with Splayed Edges to allow for Expansion

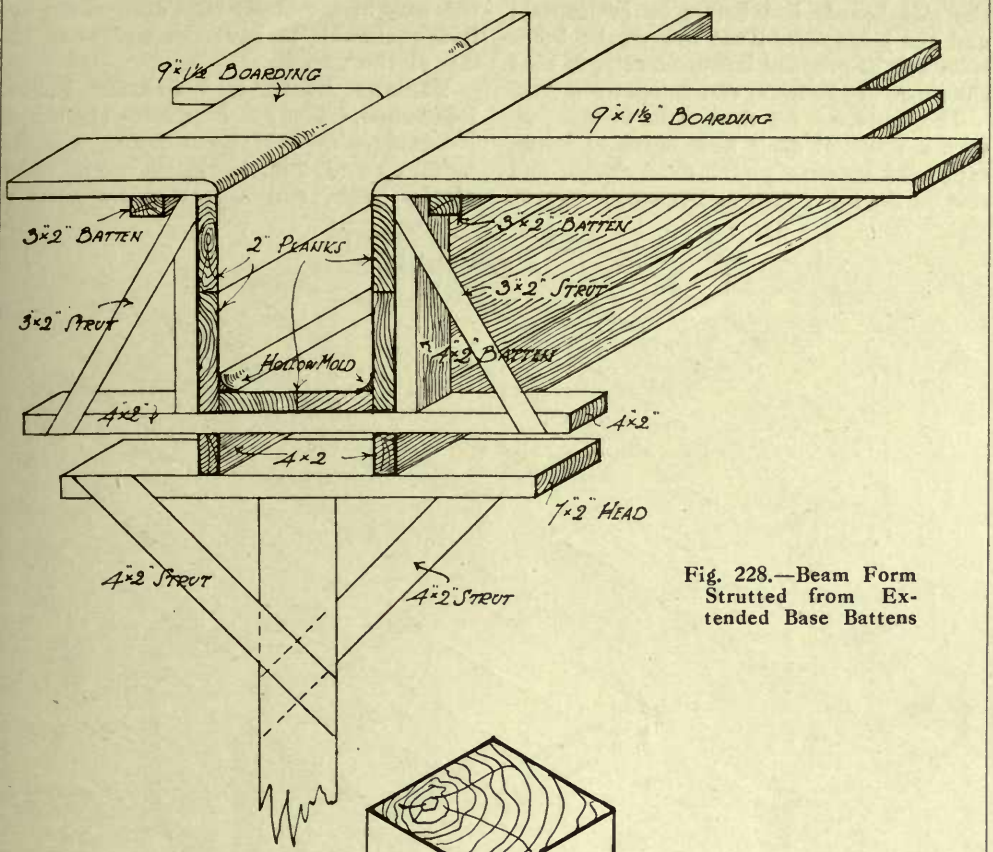


Fig. 228.—Beam Form Strutted from Extended Base Battens

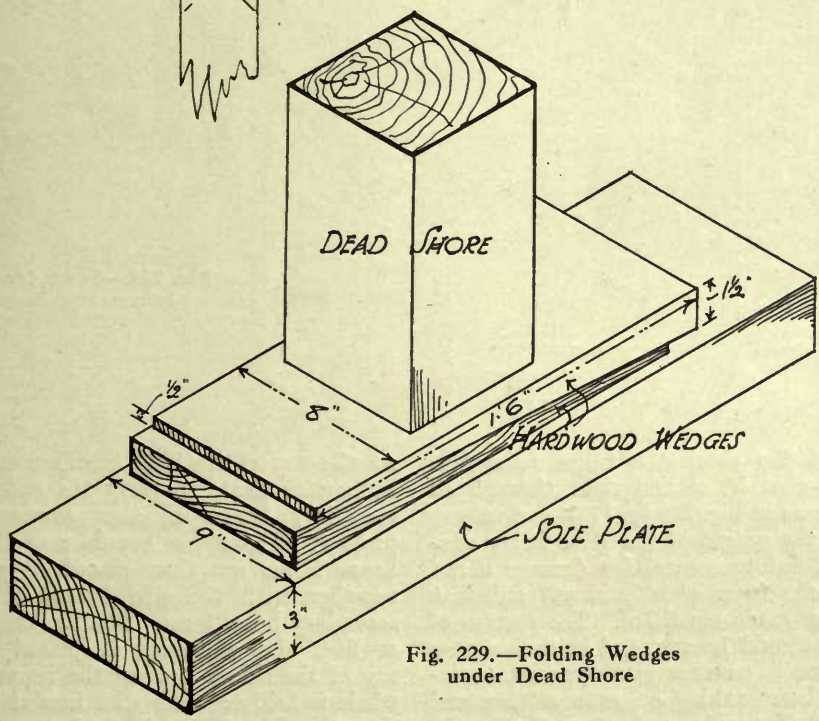


Fig. 229.—Folding Wedges under Dead Shore

fillet is better than the hollow moulding; the side boards had better be horizontal; and the joists should not rest on the beam sides but be propped independently, so that the sides, of the beam can be removed first.

The centering for floor slabs is built up with a series of deals from beam to beam, supported by head pieces, dead shores, and cills on the lower floor at frequent intervals. See that the shoring is taken to a firm bearing—that is, carried right through all the floors as in Fig. 246 (p. 176)—not only to support the weight of the floor being constructed, but to ensure that damage is not done to the floors already in yet still in a somewhat green condition. The system of wedging should be arranged in such a way that it can be eased at given periods, ensuring the beam taking an even setting and bearing. The cills, and often the heads, are provided with folding, hard-wood wedges. Fig. 229 shows the arrangement at the base of a 6-in. by 6-in. dead shore, but it is preferable to have the wedges at the top of the upright.

Various Beam and Column Forms Described.—Fig. 232 illustrates the simple Hennebique type of beam form. A is the bottom board, 2 in. thick, the exact width of the beam, and supported by posts F.

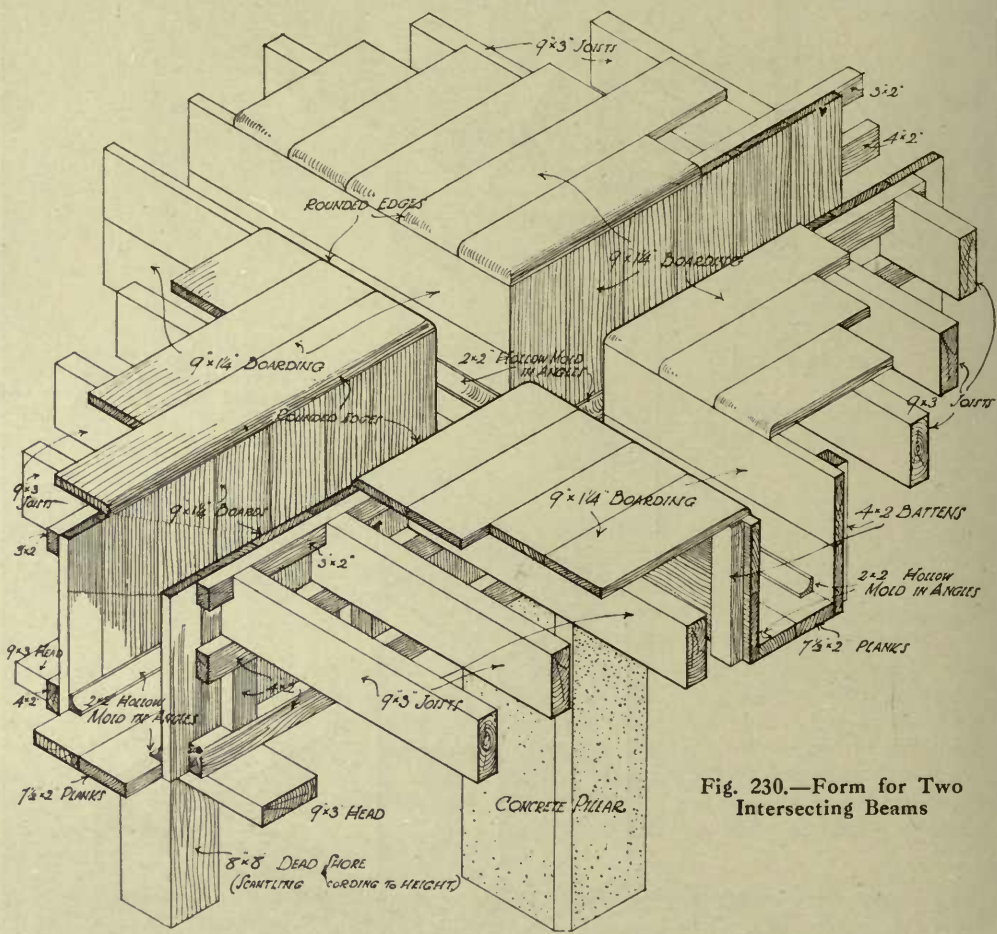


Fig. 230.—Form for Two Intersecting Beams

At the top of the side boards B are nailed battens or cleats to carry the ends of the secondary beams D, also supported by posts F. The upper boards E of the main beam forms are then placed in position, together with the side boards H of the secondary beam form. Finally, the lagging or floor slab centering is placed on the top of the side boards of the beams. The whole is held together with iron clamps, as shown in Fig. 233. At the bottom of each strut there should be hard-wood folding

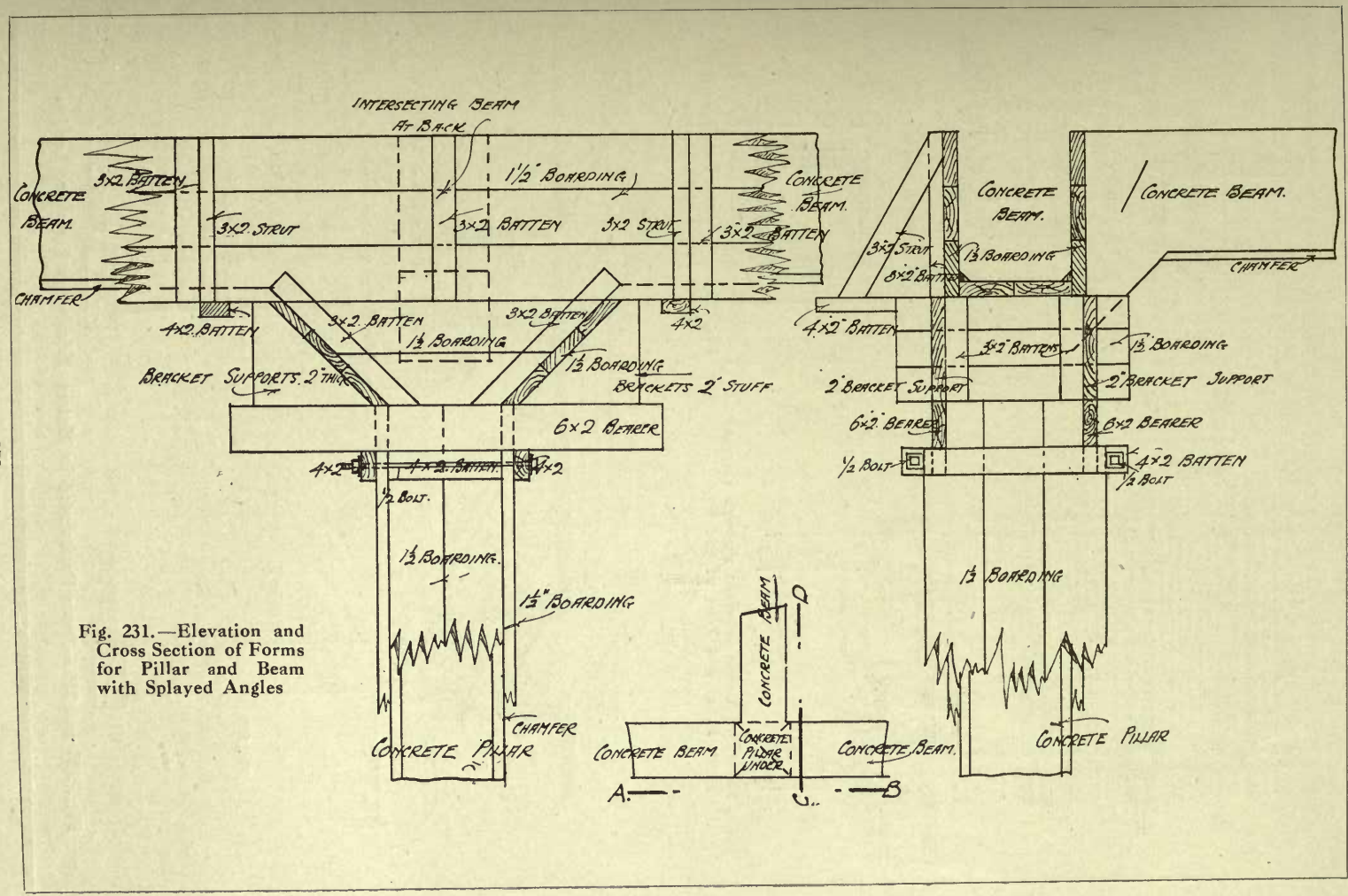
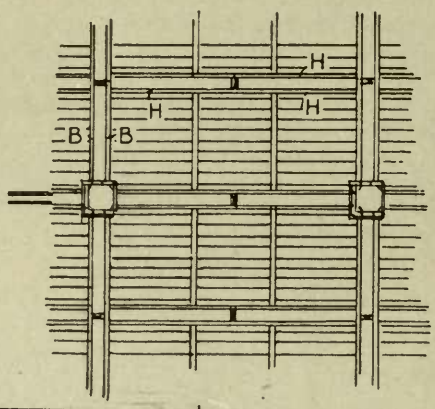


Fig. 231.—Elevation and Cross Section of Forms for Pillar and Beam with Splayed Angles



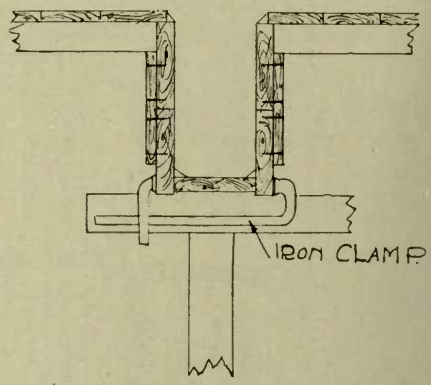


Fig. 233.—Beam Form held by Clamp

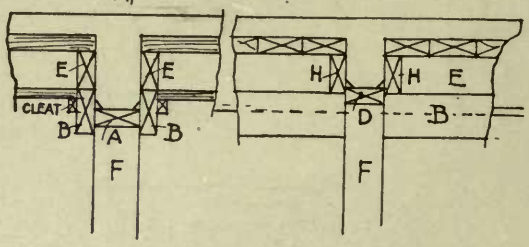


Fig. 232.—Two Cross Sections through Hennebique Floor and Plan of one Bay

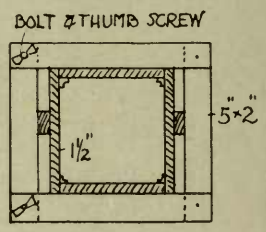


Fig. 234.—Column Form with Bolts and Thumb-screws

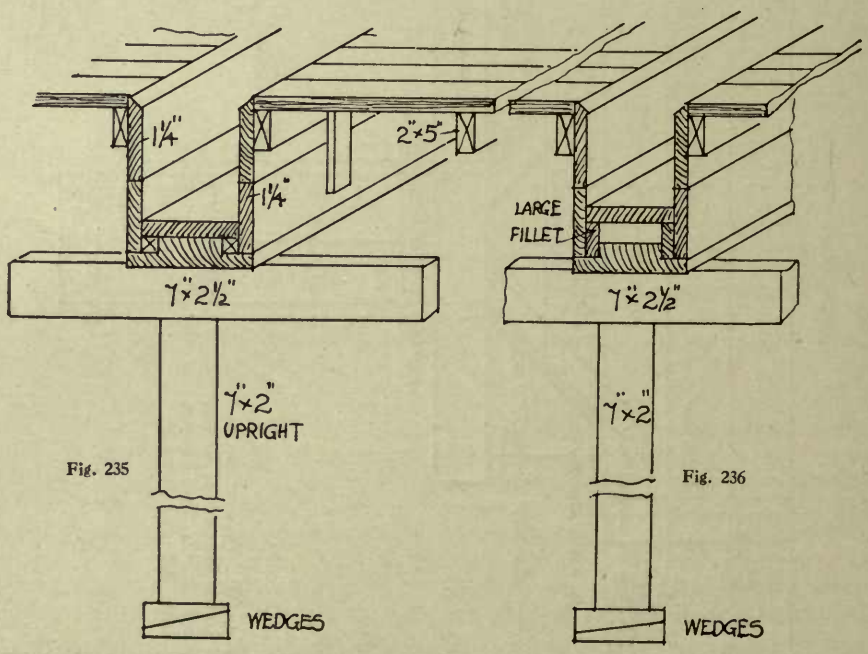


Fig. 235

Fig. 236

Figs. 235 and 236.—Beam Forms used in Messrs. Sainsbury's Premises, London

wedges, useful in setting and striking the moulds, and these should be adjusted until all is perfectly firm. If the filling in of the concrete has taken the camber, previously allowed, out of the beam, the wedges should at once be driven home, so that the camber is again obtained.

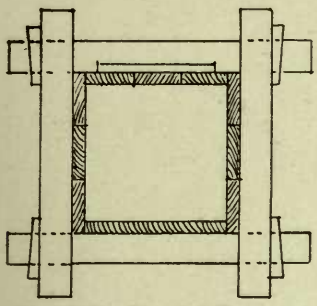


Fig. 238.—Plan of Column Form

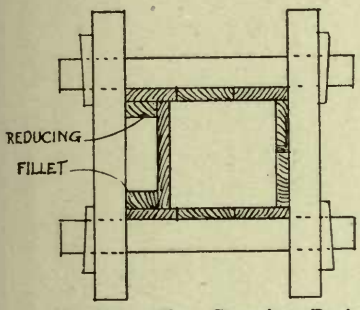


Fig 239.—Plan Showing Position of Clamps for Reduced Column Form

The beam and column forms adopted at Messrs. Sainsbury's premises in Stamford Street, London, S.E., are shown in Figs. 234 to 236. The column forms of 1½-in. vertical planed boards were built up on three sides complete. A shutter, made in two heights, of 1½-in. boards with cross battens was used for the fourth side. The whole was clamped together by 2-in. by 5-in. clamps 1 ft. 6 in. apart, adjusted at the corners with bolts and thumb-screws; the angles of the columns were ovolo moulded by the insertion of corner fillets as in Fig. 234. The columns can be reduced inside the same clamps by inserting wedges as shown. A section through the beam form is shown in Fig. 235. The point to be specially noted is the rebated support to the beam bottom. By means of fillets of various sizes placed in these rebates, the beam can be made of less breadth or depth, and so save cutting up the cheeks of the beam mould. Fig. 236 shows how the beam form may be reduced in depth and width without destroying any material.

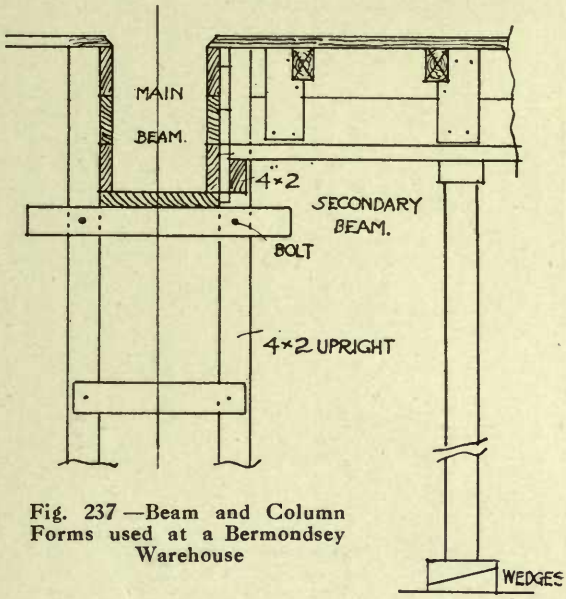


Fig. 237—Beam and Column Forms used at a Bermondsey Warehouse

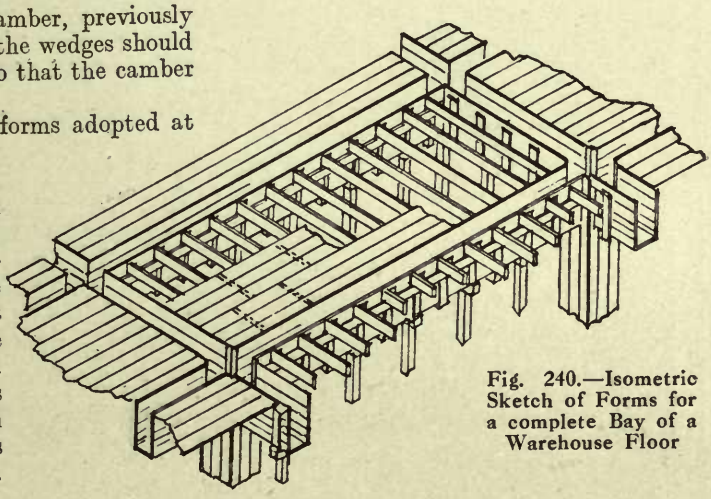


Fig. 240.—Isometric Sketch of Forms for a complete Bay of a Warehouse Floor

Figs. 237 and 239 show the beam and column forms used at Peak, Frean and Co.'s warehouse in Bermondsey. The beam

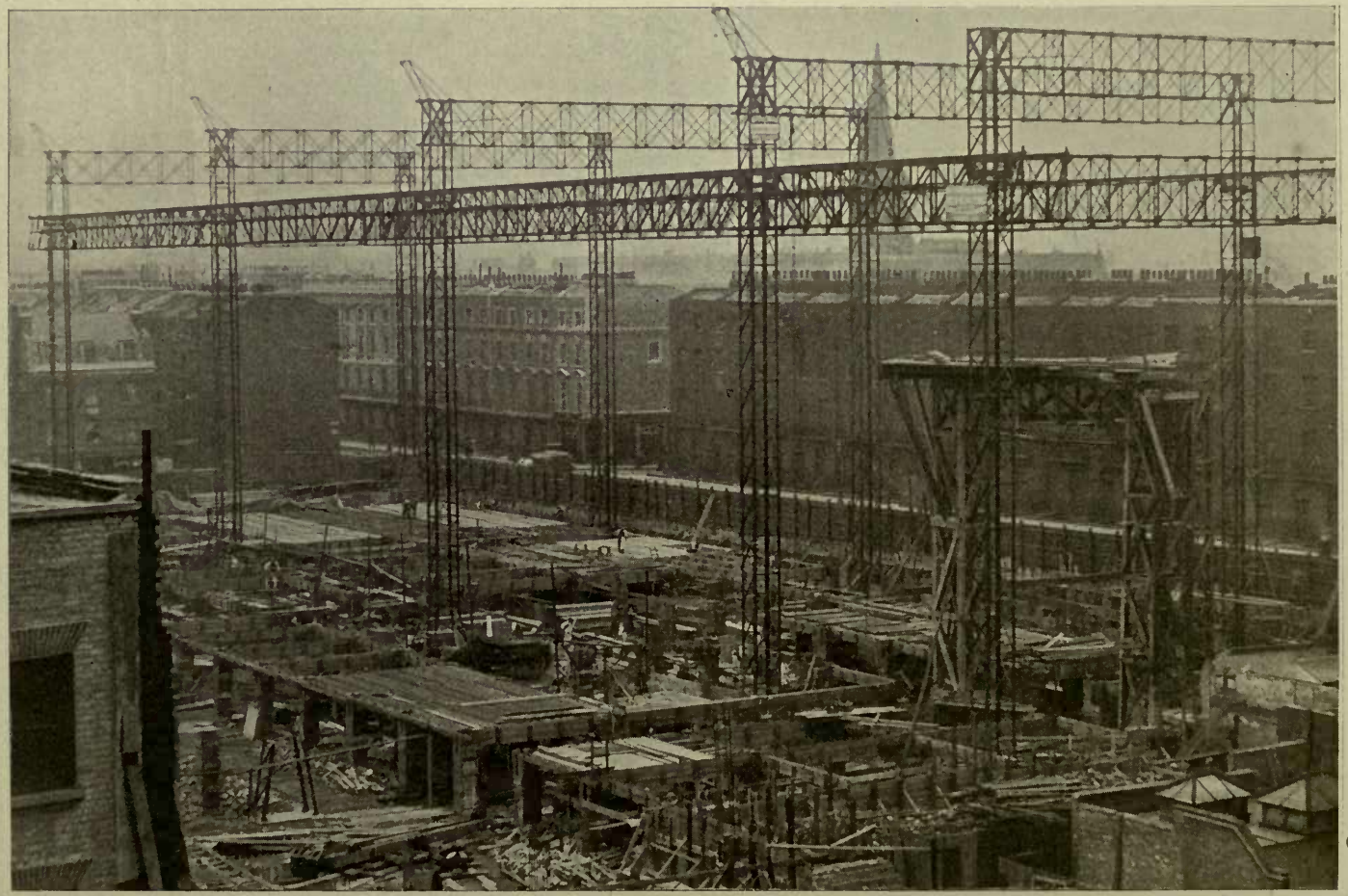


Fig. 241.—Early Stage of Forms and Centerings at H.M.'s Stationery Office

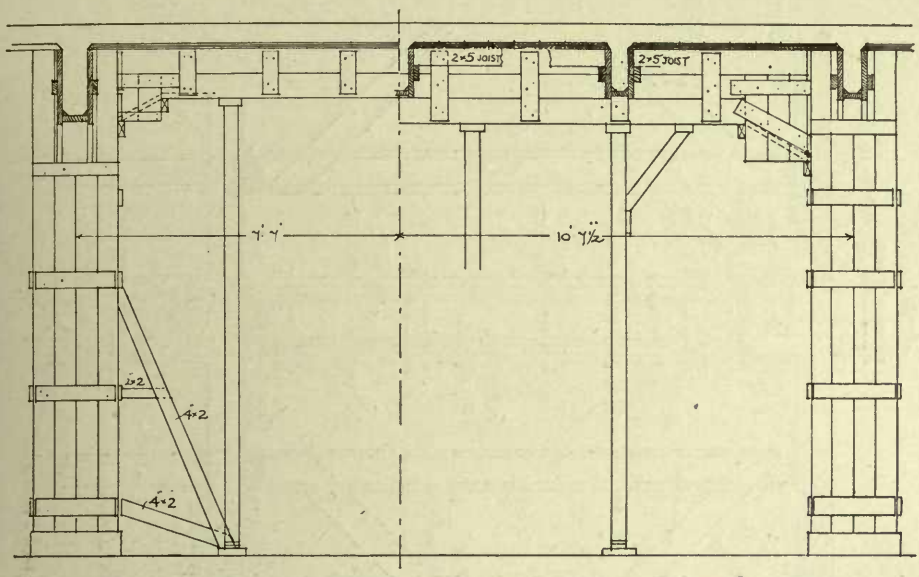


Fig. 242.—Beam and Column Forms used at H.M.'s Stationery Office

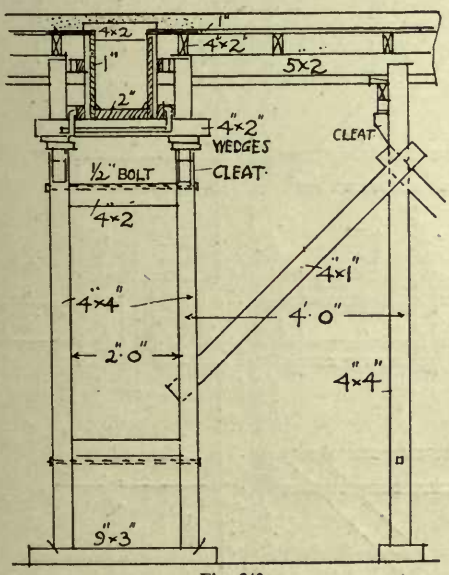


Fig. 243

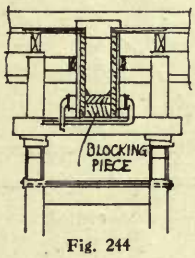


Fig. 244

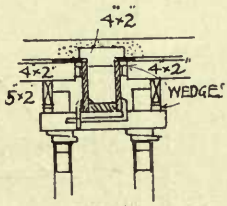


Fig. 244a

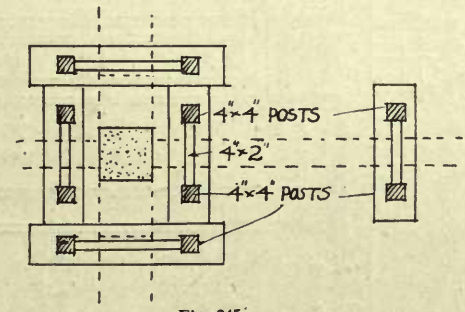


Fig. 245

Figs. 243 to 245.—Adjustable Beam Forms designed by H. Kempton Dyson

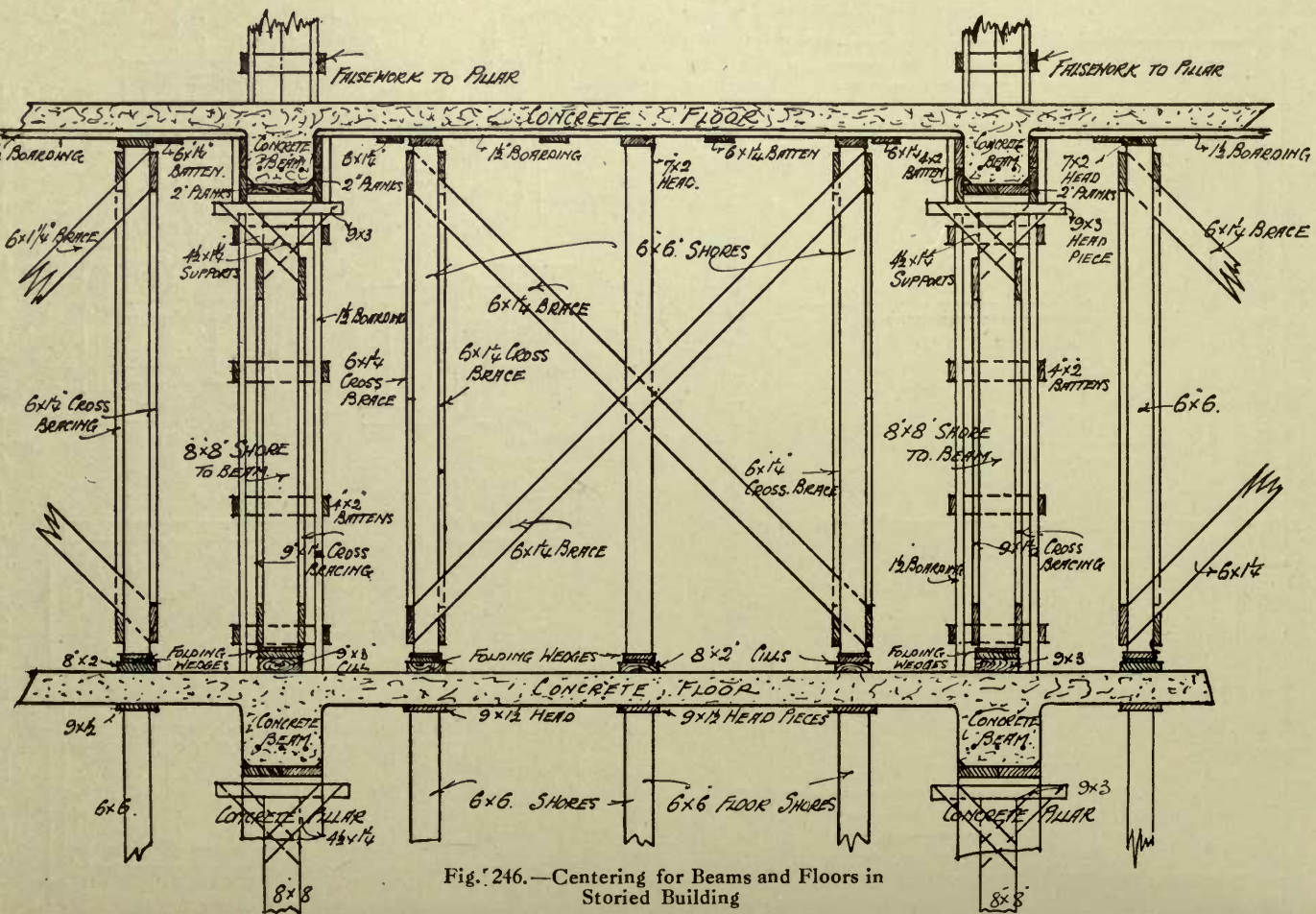


Fig. 246.—Centering for Beams and Floors in Storied Building

forms are supported by 2-in. by 4-in. uprights braced by cross battens nailed on. The top cross brace at the under-side of the beam is bolted to the uprights, and it supports the bottom board of the beams. With reference to the column forms, should it be desired to reduce the size of the column, the lower pair of clamps is brought closer together and the upper pair fitted to the grooves in the lower pair and wedged in the new position. The column can be reduced in the other direction by inserting

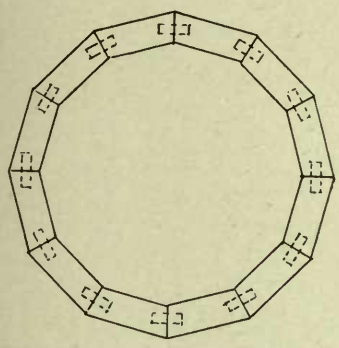


Fig. 247.—Form for Twelve-sided Column

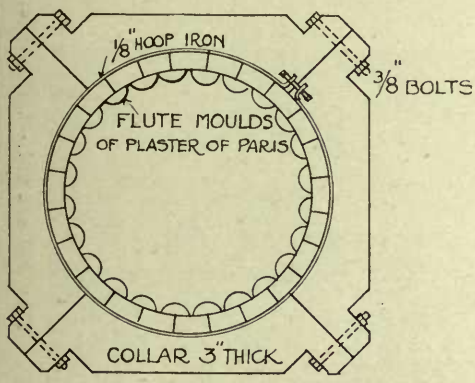


Fig. 248.—Form for Fluted Column

fillets of the proper size between the clamps and the boarding as shown. The main beam forms are supported on 2-in. by 4-in. uprights in pairs at every 3 ft. 6 in. of the length. The top cross pieces immediately under and supporting the bottom board of the beam are bolted to the uprights. The secondary beams rest on fillets nailed to the sides of the main beams and also on 2-in. by 4-in. uprights at 3-ft. 6-in. centres. The lagging of the floor slab is supported on 2-in. by 4-in. joists 2 ft. apart, the ends of the joists resting on the secondary beam forms.

Fig. 240 gives a good idea of the usual construction of the centering of floor slab, girders and secondary beams; the illustration shows a complete bay of a warehouse floor in isometric projection.

Beam and Column Forms in H.M.'s Stationery Office.—Fig. 242 shows the beam and column forms for the Government Stationery Office in Stamford Street, S.E.

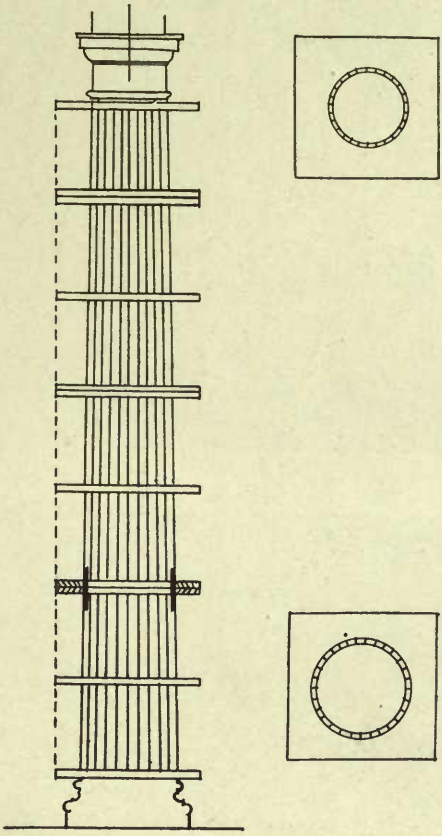
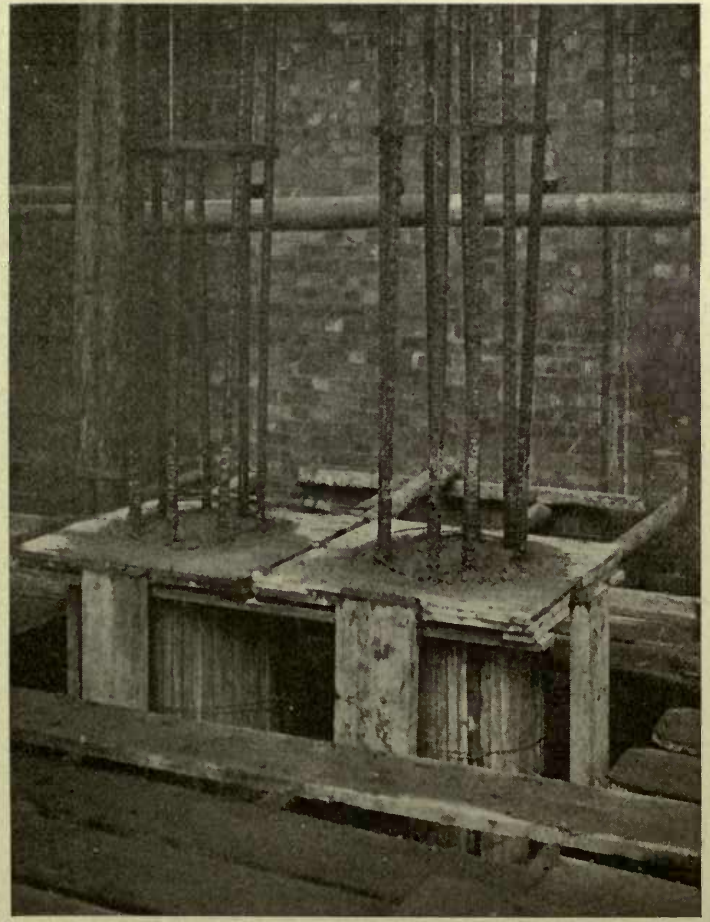
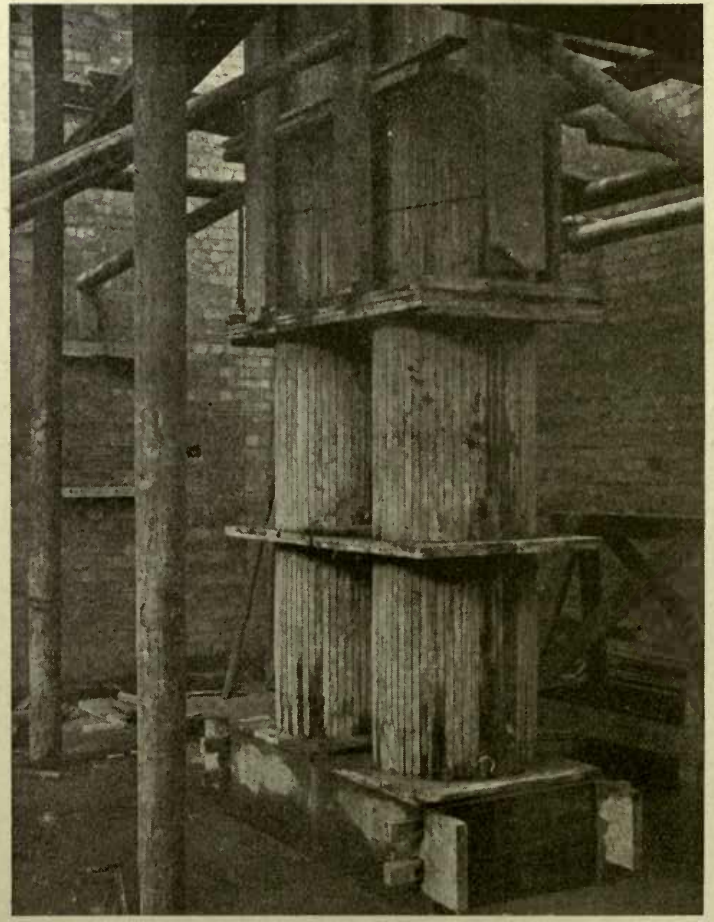


Fig. 249.—Form for Diminished Column

The main beams have a span of 21 ft. 3½ in., and the intermediate beams 15 ft 2 in. The columns are 20 in. by 20 in. There are so many bays of exactly similar dimensions that the centering could be used over and over again. The drawings explain themselves, but a point to be noted is the method of supporting the boarding for splaying the haunches of the beam on 2-in. by 4-in. fillets nailed to the column. A photograph showing much of the false-work is reproduced in Fig. 241; the steel gantry, which is a conspicuous feature in this view, was





Figs. 250 and 251.—Columns at Wesleyan Hall, Westminster

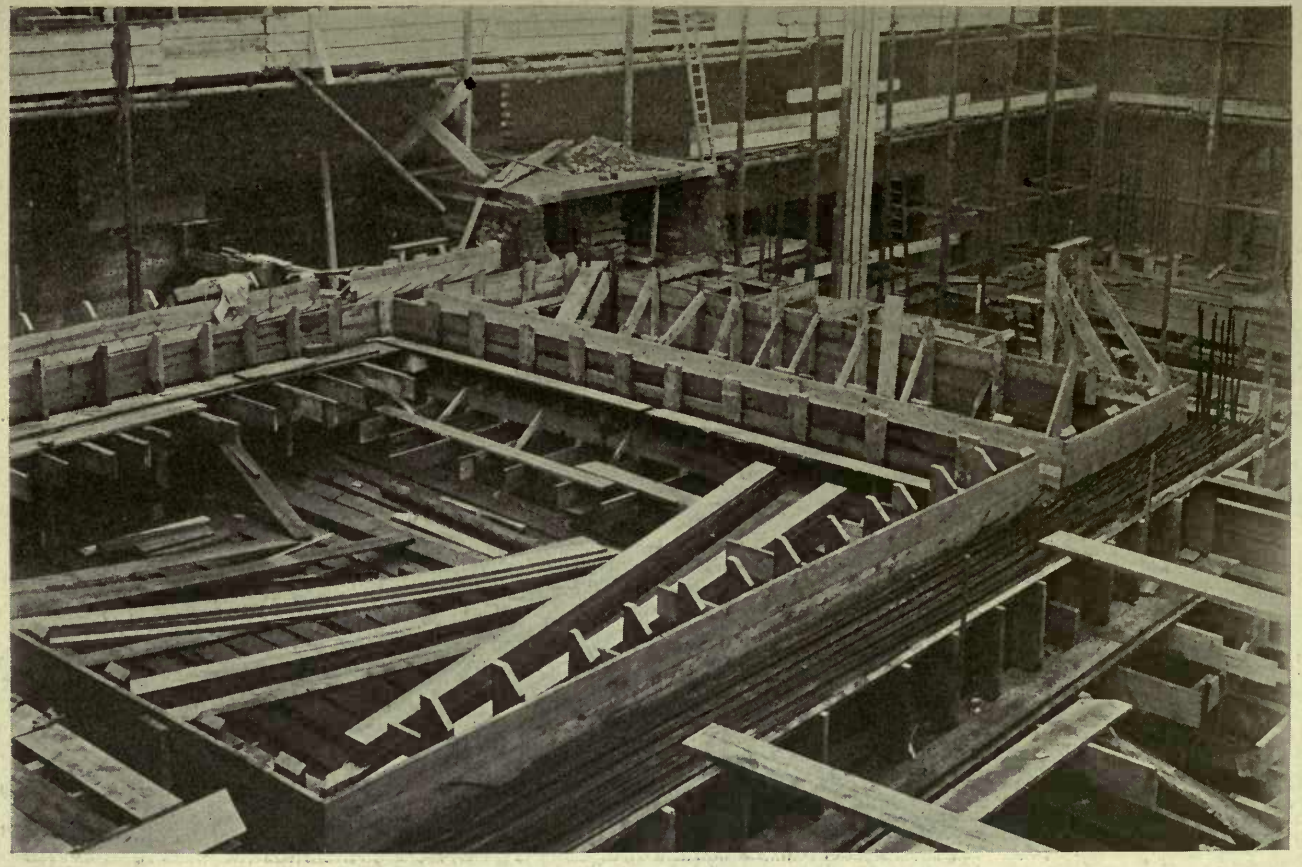


Fig. 252.—Beam Forms, etc., at Wesleyan Hall, Westminster

employed for raising the building materials, electric travelling cranes running upon it.

Beam Forms Designed by H. Kempton Dyson.—Figs. 243 to 245 show beam forms designed by H. Kempton Dyson. A point to be noted is that the beam can be reduced in depth by means of a blocking piece. The cleats on the uprights can be readjusted to any height, thus saving the cutting of the uprights to different lengths to vary the height. A different size of base-plate at the bottom of the uprights may also be employed for the same purpose. The cross bearers under the beam bottom support 2-in. by 5-in. joists, which in turn support the 2-in. by 4-in. joists carrying the floor slab boards. These joists as arranged are not the full length between the beams, and wedges are inserted between their ends and the beam boxes to keep the latter in position. The boarding carrying the slab runs parallel to the beams, and can be used in fairly long lengths, thus saving waste in cutting. The joists being supported direct from the uprights, the parts

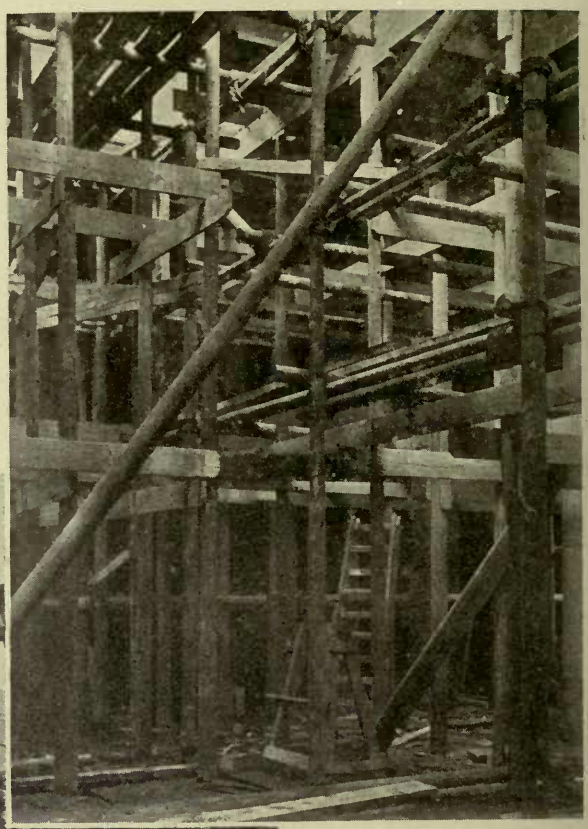


Fig. 254.—Centering for Floor at Wesleyan Hall, Westminster

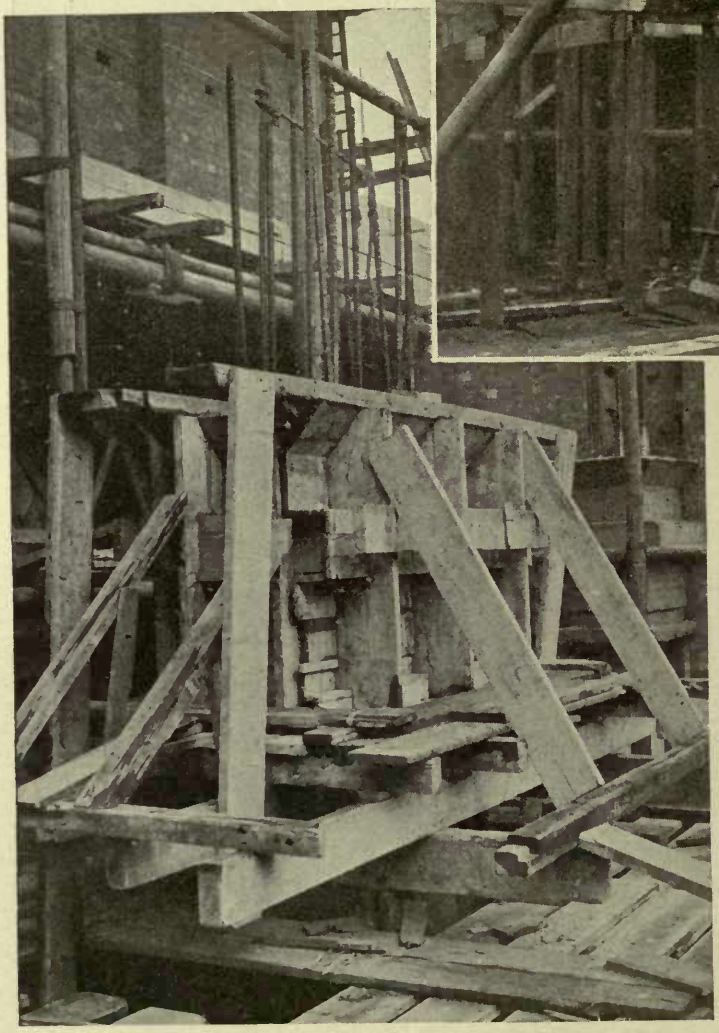


Fig. 253.—Form for Column Base at Wesleyan Hall, Westminster

of the beams, floor, and columns can be easily struck independently of each other.

Various Column Forms.—The form for

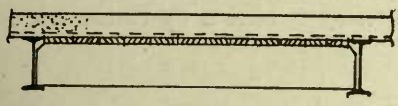


Fig. 255.—Centering Resting on Flanges of Steel Joists

a twelve-sided column, illustrated by Fig. 247, is made of wooden staves dowelled together and hooped with adjustable iron straps. The form for a fluted column, shown in Fig. 248, consists of narrow laggings dowelled together and hooped round with adjustable

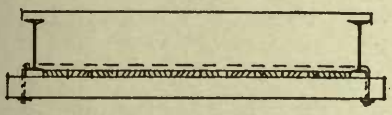


Fig. 256.—Centering Suspended from Flanges of Steel Joists

iron bands. The flutes are formed of plaster-of-paris attached to the inside by screws which are inserted from the outside; in removing them, the screws are drawn and the plaster flutes remain as a protection after striking until the building is completed.

In the new Wesleyan Hall at Westminster circular columns with entasis are used. The forms (Fig. 249) were made in sections 4 ft. high, the narrow vertical laggings being easily bent to the required curve and secured to horizontal

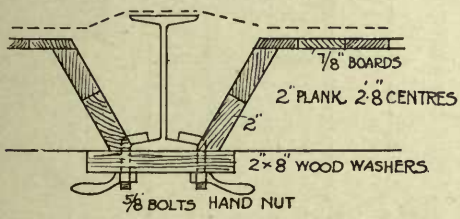


Fig. 257.—Centering for Concrete Floor having Steel Main Beams

pieces set out with their outer edges plumb, while their inner edges coincided with the diameter of the column; these horizontal pieces were fixed at intervals of 2 ft. The method gave such good results that the columns could be finished with $\frac{1}{8}$ in. of cement. Figs. 250 to 254 are photographic views showing the false-work to beams and columns.

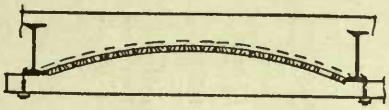
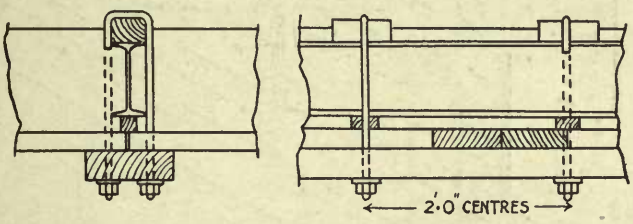


Fig. 258.—Centering for Arch Ceiling between Joist Flanges

CENTERINGS FOR CONCRETE FLOORS CARRIED BY STEEL JOISTS

The simpler varieties of centering for steel joists will now be briefly dealt with. Fig. 255 shows a concrete slab lying on the top flanges of rolled steel joists; timber bearers rest on the bottom flanges and support the lagging. Fig. 256 shows a similar centering supported on the bottom flanges by means of hook hangers or suspended by slings from the top. It may be arched as shown by dotted lines. Fig. 257 shows a design by W. F. Kearns for a reinforced concrete floor with steel main beams. The defect of these three floors is that, even



Figs. 259 and 260.—Floor Centering supported by Hangers

although the centering is kept low, the bottom flanges of the steel joists are not embedded in the concrete, and therefore are exposed to the effects of fire; the defect is obviated by having a ceiling on iron hangers, metal lathing being embedded in the plaster. Fig. 258 shows the centering for an arch ceiling between the joist flanges, suspended by $\frac{5}{8}$-in. bolts. In the method shown by Figs. 259 and 260, square-headed hooked bars are hung on runners which are placed on the upper flanges of the steel joists, the bars being placed alternately on each side of the joist.

WALL AND PARTITION FORMS

The shuttering for walls consists of battened widths passed down behind quartering driven into the ground and kept apart by iron distance pieces at the top, the

arrangement for battering the face being obtained by sloping the uprights to the required angle. In the case shown by on it, to which the uprights, spaced at not more than 5-ft. centres, are fixed and braced by runners and struts to keep the horizontal boards as rigid as possible. The outside shuttering is carried up to the full height

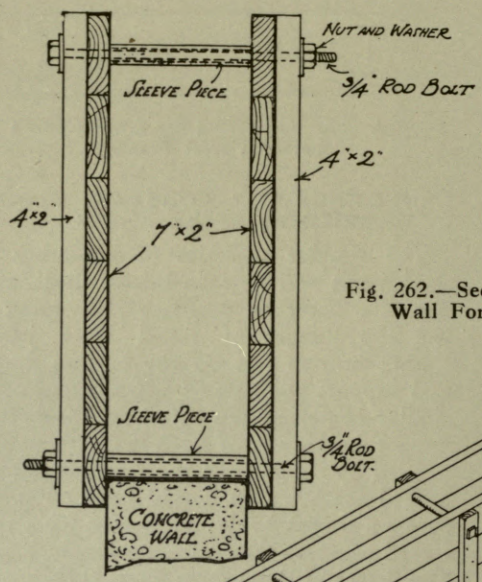


Fig. 262.—Section of Wall Form

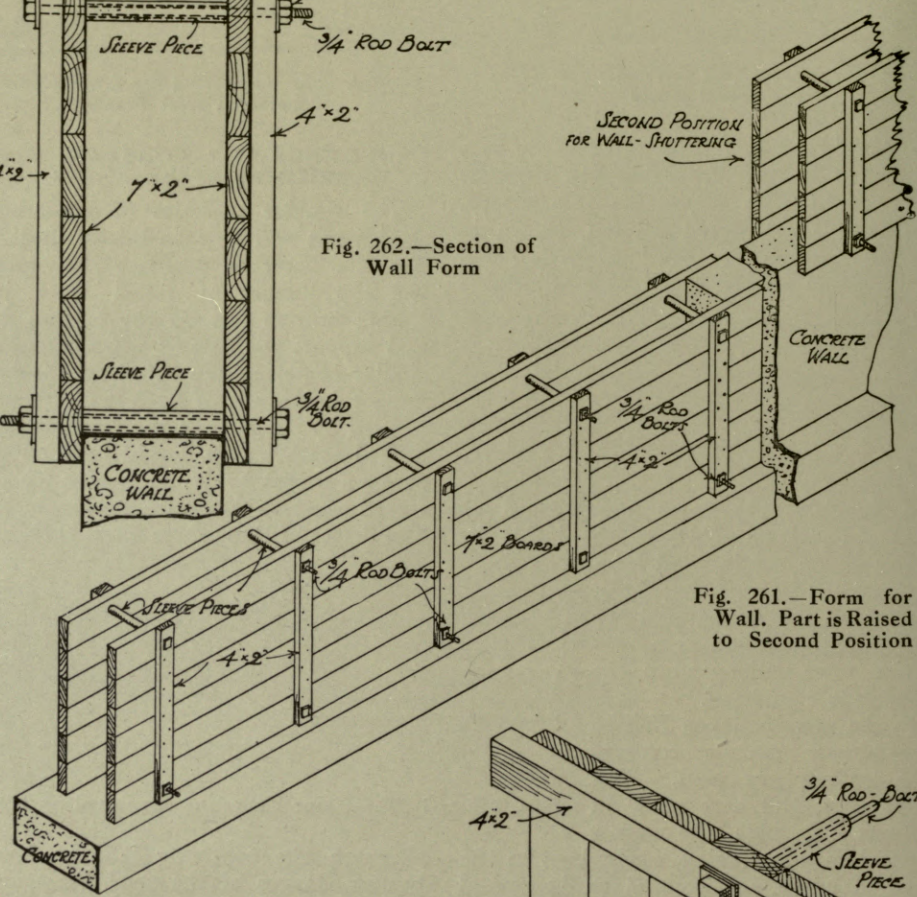


Fig. 261.—Form for Wall. Part is Raised to Second Position

Fig. 261 shutters are made of 7-in. by 2-in. planks with 4-in. by 2-in. ledgers, and by employing sleeve pieces over the $\frac{3}{4}$-in. rod bolts fitted with heads, nuts, washers, etc., the sides of the form are kept the required distance apart. In the right-hand part of the illustration a section of the wall is shown complete, and the shutter has been raised to the second position resting on the sleeve pieces, as further shown in Fig. 262. In the next illustration (Fig. 263) an alternative, but inferior, method is shown, the 6-in. by $1\frac{1}{2}$-in. boards having 4-in. by 2-in. continuous head and cill pieces.

In the type of form illustrated in Figs. 264 and 265 it will be seen that the floor slab having first been laid, cill pieces are placed

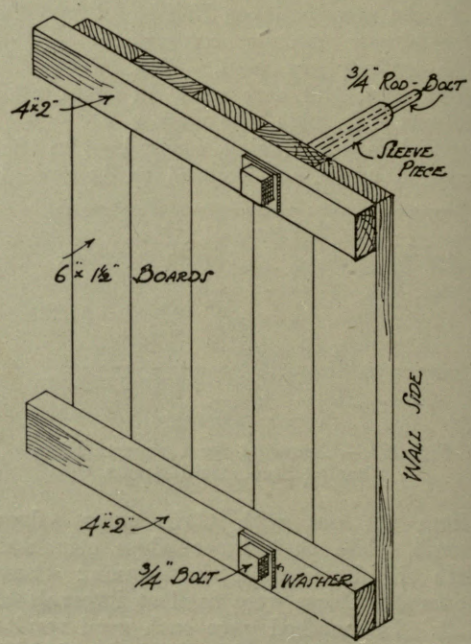
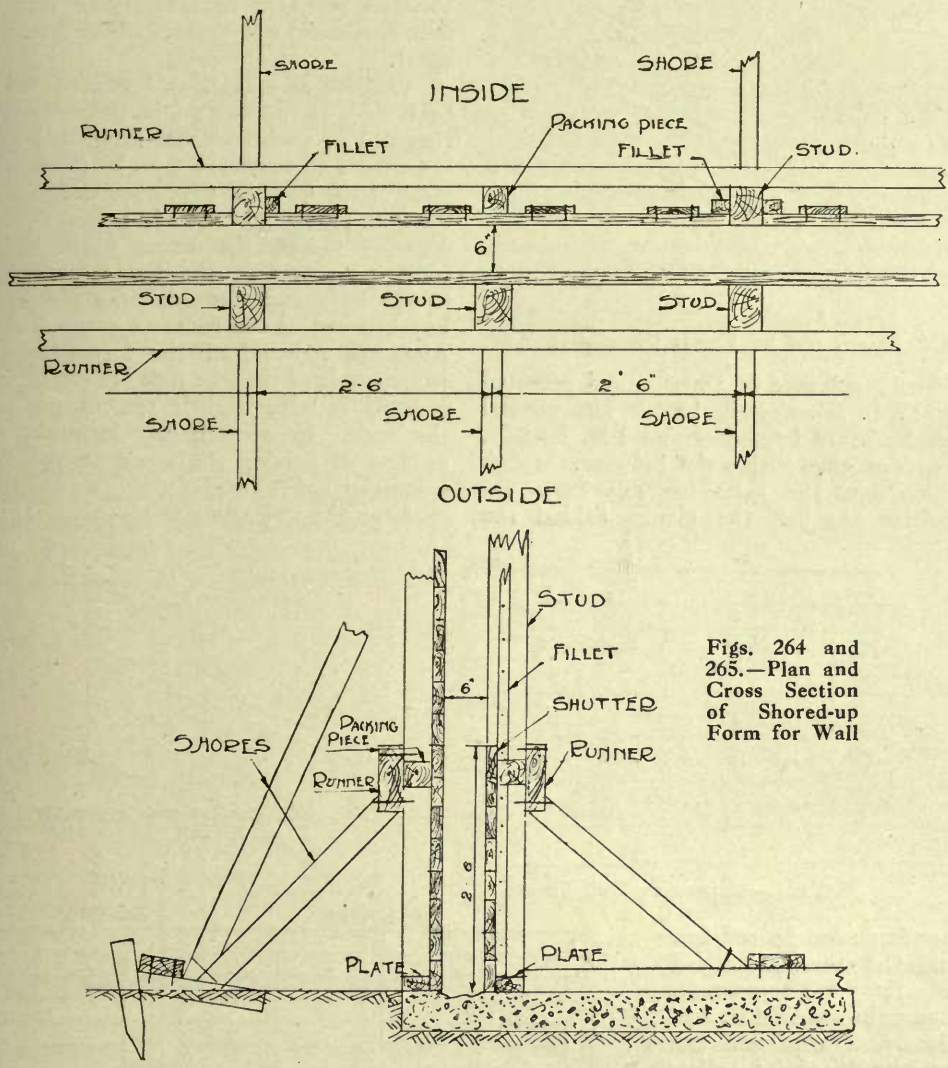


Fig. 263.—Panel for Wall Form

the inner shuttering is made in panels 5 ft. wide and 2 ft. 6 in. high, and built up as the work proceeds. The size of the uprights is governed by their distance apart, the height and thickness of wall, and amount of strutting. This design is not recommended.

For walls from 6 ft. to 14 ft. high, 2-in. by 7-in. or 3-in. by 6-in. uprights may be used if not placed more than 2 ft. 6 in. apart or 2 in. by 5 in. if spaced not more than 2 ft. apart. The uprights should be stayed by one or two rows of horizontal runners 2 in. by 7 in., or flooring boards propped up by 2-in. by 4-in., 2-in. by 5-in., or 3-in. by 4-in. struts, whichever scantling is at hand. It follows that if 2-in. shuttering is used the studding can be placed farther apart, and of course made correspondingly heavier in scantling.

Fig. 266 shows the false-work for a simple foundation wall. First, 2-in. by 6-in. horizontal bearers are laid to 2-in. by 4-in. uprights, the latter having pointed ends which are driven into the soil; the bearers are kept in position by being secured to stakes driven into the ground. The uprights are braced and kept the proper distance apart by 1-in. by 4-in. battens nailed to their tops, vertical packing pieces $\frac{7}{8}$ in. square being placed between the studs to stay the shuttering; after a section of the concrete has been finished, the boards are raised to



Figs. 264 and 265.—Plan and Cross Section of Shored-up Form for Wall

continue the wall above. The footing is formed by allowing the first layer of concrete to flow under the bottom boards.

Ransome's Wall Form.—Fig. 267 shows

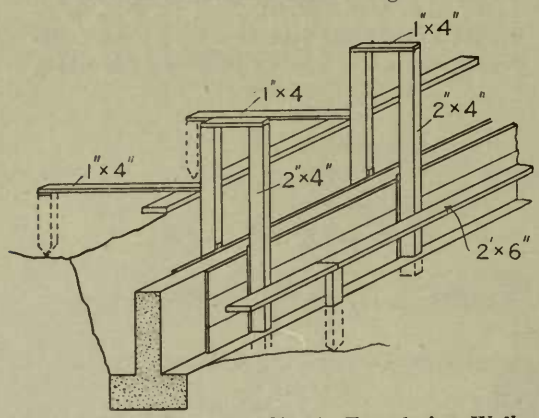


Fig. 266.—Form for Simple Foundation Wall

a form much used in America, and patented by E. L. Ransome in 1885. The vertical standards are formed of two 1-in. by 6-in. boards on edge with a slot between, through which pass the bolts (*see* Fig. 269). By undoing the bolt the planks behind the standards can be set free and the standards raised as the wall proceeds. The walls are in 4-ft. sections. A core box can be introduced for hollow walls.

The construction of a large panel shutter for walls is shown by Fig. 268. The panels are made of 2-in. by 8-in. plank dressed one side. The panels are erected by placing the bottom row in position and fixing them by stay lasts. Then another row is placed on top so that the vertical joints in each row coincide. Long 6-in. by 6-in. timbers are next placed vertically at the joints, just fitting in the spaces left between, the short 6-in. by 6-in. ledges, being 3 in. from the ends of the form. Other rows are added similarly to suit the height of the wall. These long 6-in. by 6-in. verticals are kept in position by 6-in. by 6-in. horizontal wales placed about 2 ft. 6 in. apart.

This set of operations is carried out on both sides of the wall; $\frac{5}{8}$-in. diameter rods, threaded at both ends and fitted with nuts, are passed through the 6-in. by 6-in. wales, clear of the 6-in. by 6-in. uprights, through the 2-in. planking every 4 ft.; 2-in. by 4-in. distance pieces, the exact width of the wall, are placed in the forms quite close to the bars, and these distance pieces are knocked out as the concrete reaches them. After the removal of the forms the projecting ends of the iron rods are cut off, and covered in with a little cement put over the ends. In a particular instance, this method of holding the forms proved very successful, not the slightest bulging occurring although the wall was a thick one.

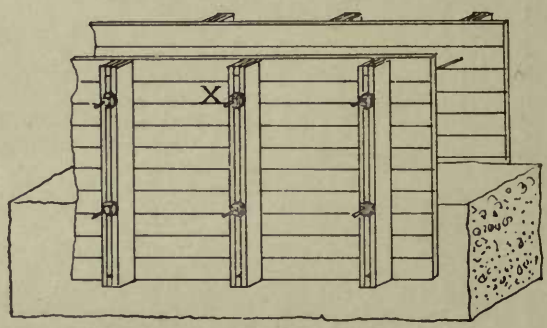


Fig. 267.—Ransome's Wall Form

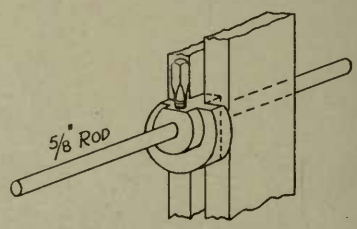


Fig. 269.—Collar and Set-screw at X (Fig. 267)

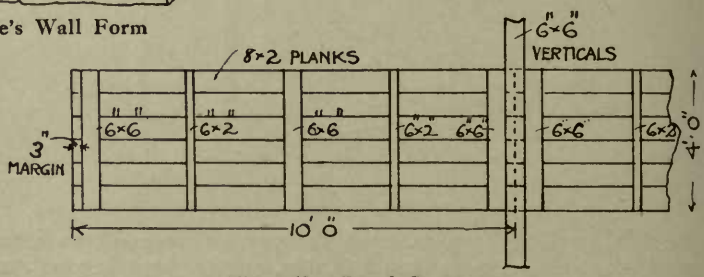


Fig. 268.—Panel Shutter

IMITATING MASONRY WALLS

A method adopted by E. L. Ransome is to fasten wood V strips to the wall forms, so as to produce an imitation of masonry (*see* Fig. 270). The surface may be left

plain or rusticated by the application of a pick or chisel.

STAIRCASE FORMS

The forms on which the stairs at Peek, Frean and Co.'s warehouse at Bermondsey

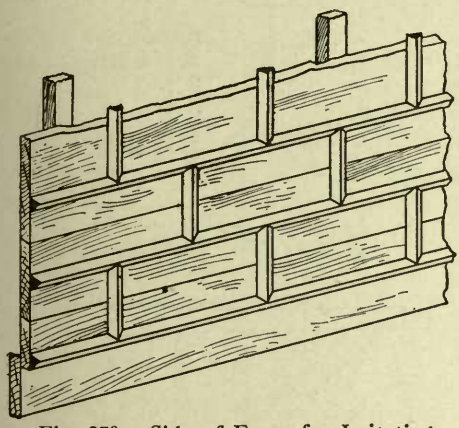


Fig. 270.—Side of Form for Imitating Masonry Wall

were made are shown by Figs. 271 to 273. They are in continuous flights, with half-space landings. The stringers under the soffit of the stair are 2 in. by 6 in., and the carriage 2 in. by 4 in., with cross stemming, as shown in the isometric sketch. The stringers are supported at intervals by 2-in. by 5-in. uprights; where the uprights require to be more widely spaced, they are 3 in. by 6 in. Horizontal ledgers 2 in. by 5 in. are fixed to stay the uprights and stringer. The soffit of the steps is formed by flooring boards being nailed to the stringer. The form for the steps themselves is constructed of triangular bracket pieces nailed to the flooring with a concave moulded fillet to form the nosing to the riser (*see* Fig. 273).

In many cases, stairs are made in advance and erected similarly to stone hanging steps. In that case the stringer is moulded like an inclined beam properly reinforced top and bottom with rods, and rebated on the lower edge to receive the steps. An examination of the form for the steps will show at a glance that the usual back check in hanging steps is got by fixing in triangular fillets.

METAL FORMS FOR WALLS, ETC.

A recent system of metal forms for walls, columns, and girders is that of the Blaw Steel Centering Co., and it is designed to combine economically the three processes of form erection, namely, the assembling of forms, lining them, and spacing them. The panels shown in Fig. 274, which make up the wall forms, are based on a standard

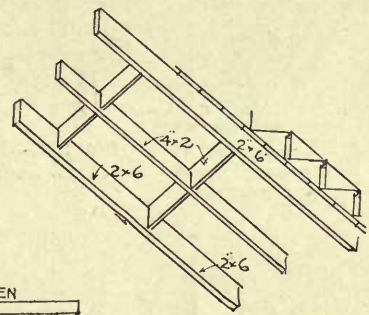


Fig. 272.—Stringers and Carriage of Stairs

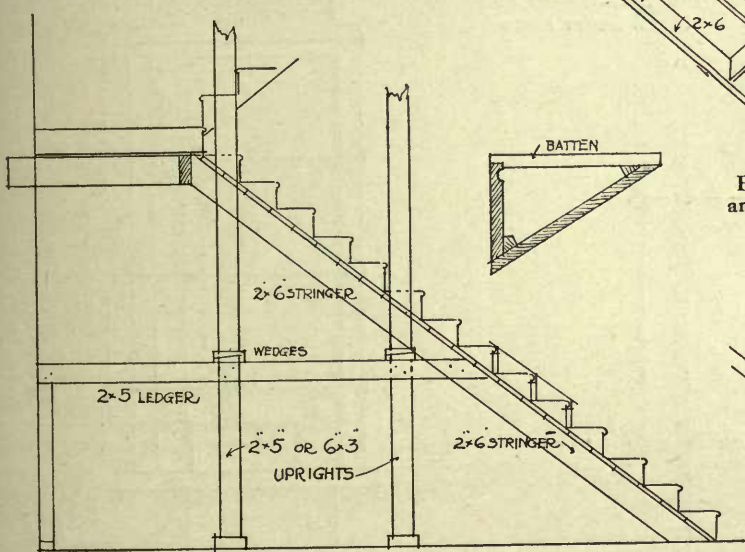


Fig. 271.—Form and Centering for Staircase

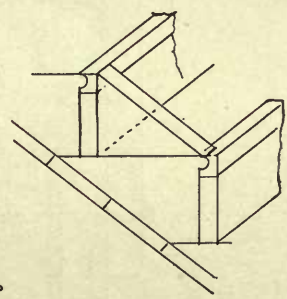


Fig. 273.—Form for Steps

sized surface of 24 in. square. Forms are furnished in fractional sizes 25 in. by 12 in., 24 in. by 6 in., 12 in. square and 6 in. square. These sizes enable the contractor to work to any dimensions which are a multiple of 6 in. Corner panels are supplied with returns 12 in. on each side and 24 in. high. The panels have angle flanges on all four sides, these being spot welded to sheet steel plates. There are no rivet marks on the face of the forms.

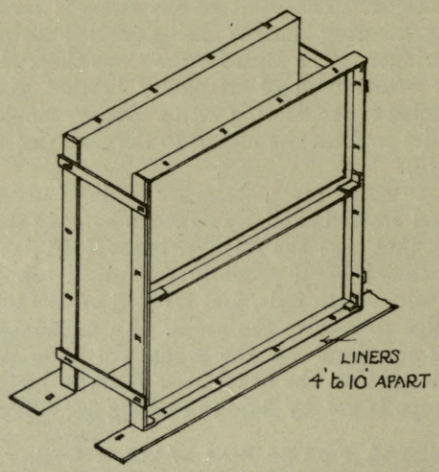


Fig. 274.—Metal Panel Form for Walls

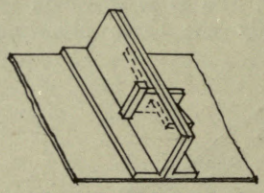


Fig. 275.—Method of Fastening Panel Flanges together

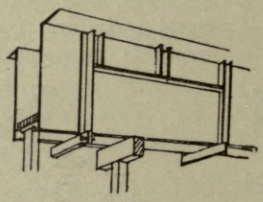


Fig. 276.—Detail of Metal Beam Form

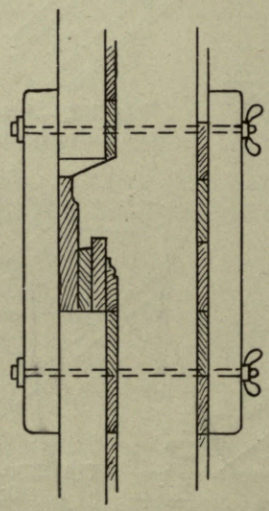


Fig. 277.—Ransome Form for Cornice

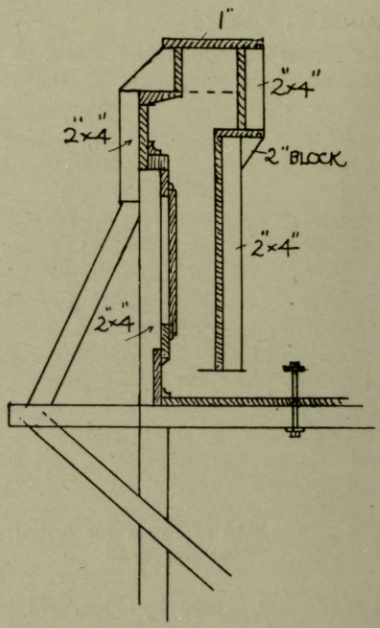


Fig. 278.—Form for Spandrel Wall to Bridge

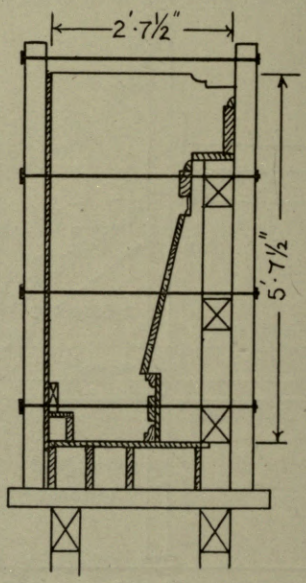


Fig. 279.—Form for Curtain Wall with Moulded Cornice

The plates are joined by special fastenings passing through slots in the flanges. The flanges are fastened together as shown in Fig. 275. Adjustable beam and girder moulds are also made, using wooden planks

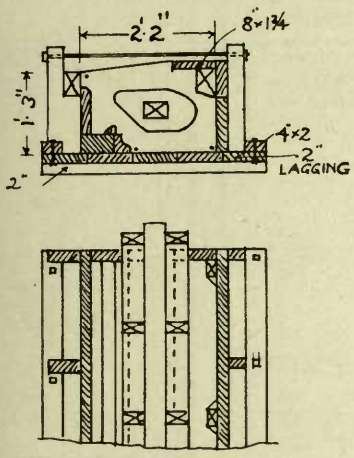


Fig. 280.—Two Sections of Form for Cornice to Hollow Coping

as the bottoms. These pieces are so clamped together that the sides may be removed and used in another part of the work, while the bottom board form remains in place securely shored until the curing of the concrete is complete (*see* Fig. 276).

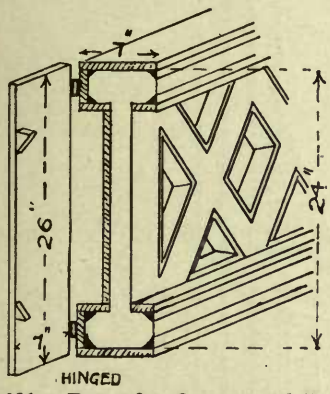
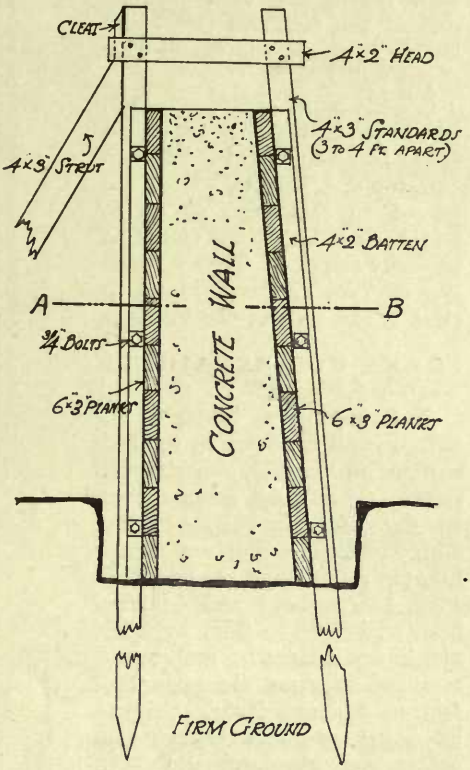
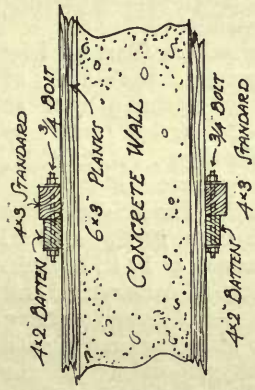


Fig. 281.—Form for Ornamental Parapet

FORMS FOR CORNICES, MOULDINGS, ETC.

Fig. 277 shows Ransome forms applied to a cornice. The false-work for a spandrel wall to a bridge is shown by Fig. 278. The shuttering is 1½ in. thick, kept in position by 2-in. by 4-in. studs with block pieces. The mouldings are made by inserting splayed and ovolo or cavette moulded slips in the angles of the form. A detail of the form for a curtain wall with moulded cornice and frieze is illustrated in Fig. 279, while Fig. 280 represents forms for the cornice to a hollow coping.





Figs. 282 and 283.—Vertical and Horizontal Sections of Form for Battered Retaining Wall

FORMS FOR ORNAMENTAL PARAPET

Fig. 281 shows the form for an ornamental parapet, 2 ft. high, to a bridge or balus-

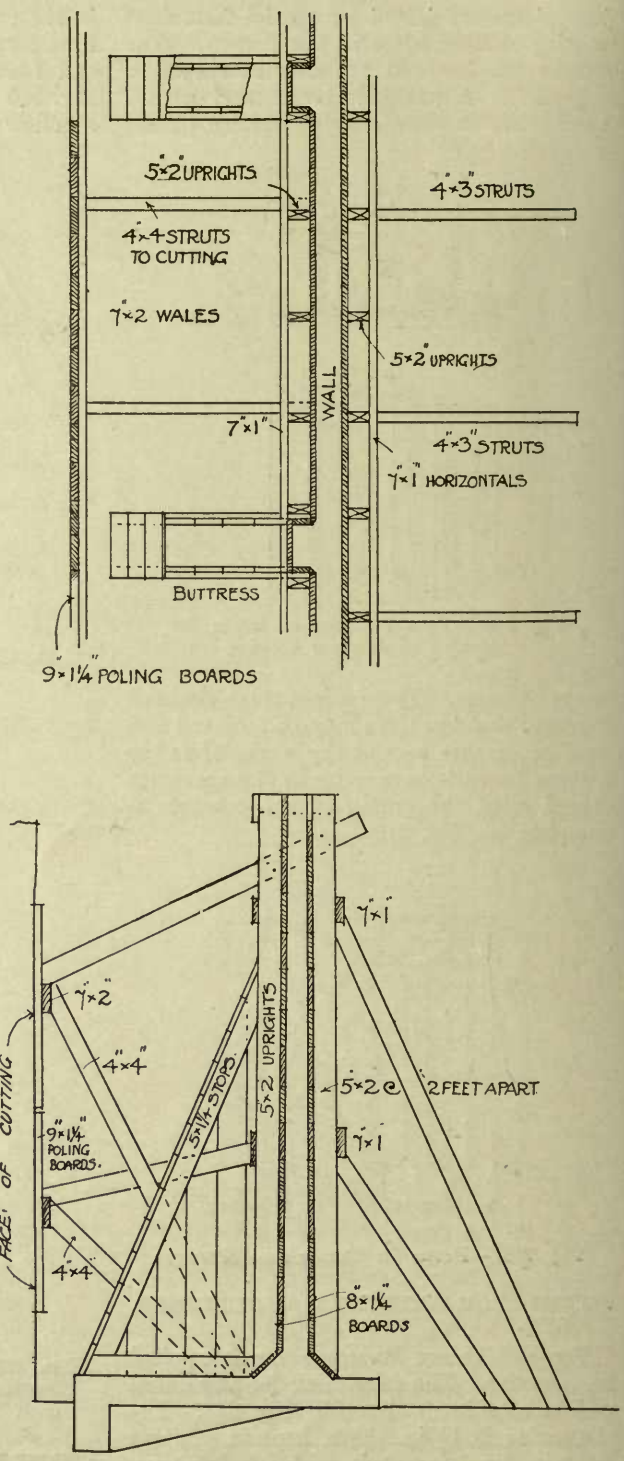
trading. Two boards 7 in. wide, corresponding to the length of the panel, one each for the top and bottom, lap over the side board, which is 6 in. wide. Two 4-in. boards are used for one side of the rail, and two $2\frac{1}{2}$-in. boards to mould the inside edge of the rail.

For the panel centre two 8-in. boards joined together are nailed to the two 4-in. boards. The two 7-in. top and bottom boards are hinged to this form so as to fold back to allow the mould to be easily emptied. The two ends are of 7-in. timber, hinged to close in the ends of the forms.

FORMS FOR RETAINING WALLS

A form for a battered retaining wall is shown in vertical section by Fig. 282, a horizontal section on line A B being shown by Fig. 283. In this, 4-in. by 3-in. standards are driven firmly into the ground and are provided with a cross-head and 4-in. by 3-in. struts. The 6-in. by 3-in. planks are battened together and arranged to come alongside the 4-in. by 3-in. standards. Centre-bit holes are made through the batten and standard, and $\frac{3}{4}$-in. bolts, 9 in. long, passed through and tightened up.

Other forms are illustrated by three examples, the first of which is a retaining wall at Bridlington, carried out by Ernest Matthews, A.M.I.C.E., and clearly illustrated by Figs. 284 and 285. The height of the wall varies from 10 ft. to 13 ft. with buttresses at 10 ft. centres. The cutting is excavated, and $1\frac{1}{4}$-in. by 9-in. rough poling boards are placed against it with two rows of wales in the height. These are shored as shown with 4-in. by 4-in. struts. The foundation slab having been put in, the 2-in. by 5-in. uprights, at about 2-ft. centres, are erected on the foundation slab and spaced apart sufficiently



Figs. 284 and 285.—Plan and End Elevation of Form and Centering for Retaining Wall at Bridlington

to allow of the thickness of the wall and $1\frac{1}{4}$ in. of shuttering on each side. The uprights are held in position by two rows of 1-in. by 7-in. flooring nailed on and strutted with 3-in. by 4-in. raking and horizontal struts to the cutting as shown, with vertical flooring boards cut to the rake and nailed to $1\frac{1}{4}$-in. by 5-in. stops, which are secured to the uprights. The forms for the buttress should be boarded in on top, ahead of the concrete laying, with 1-in. by 7-in. flooring on the slope. The forms are wetted both inside and out before any concrete is placed in them. A 2-in. by 7-in. board, acting as a wind strut, is nailed to the top of the uprights and the waling at the cutting.

Retaining Wall at Local Government Offices.—The forms for the retaining wall at the

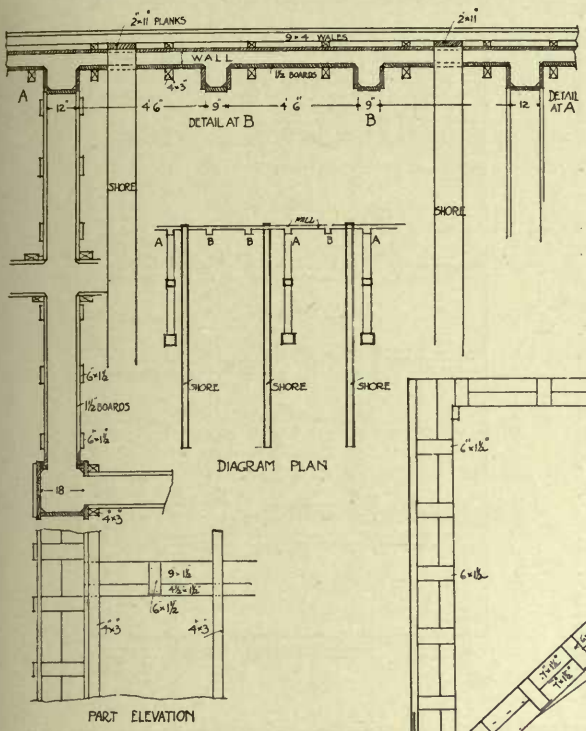


Fig. 287.—Diagram, Plan, and Details of Centering for Retaining Wall at Local Government Offices

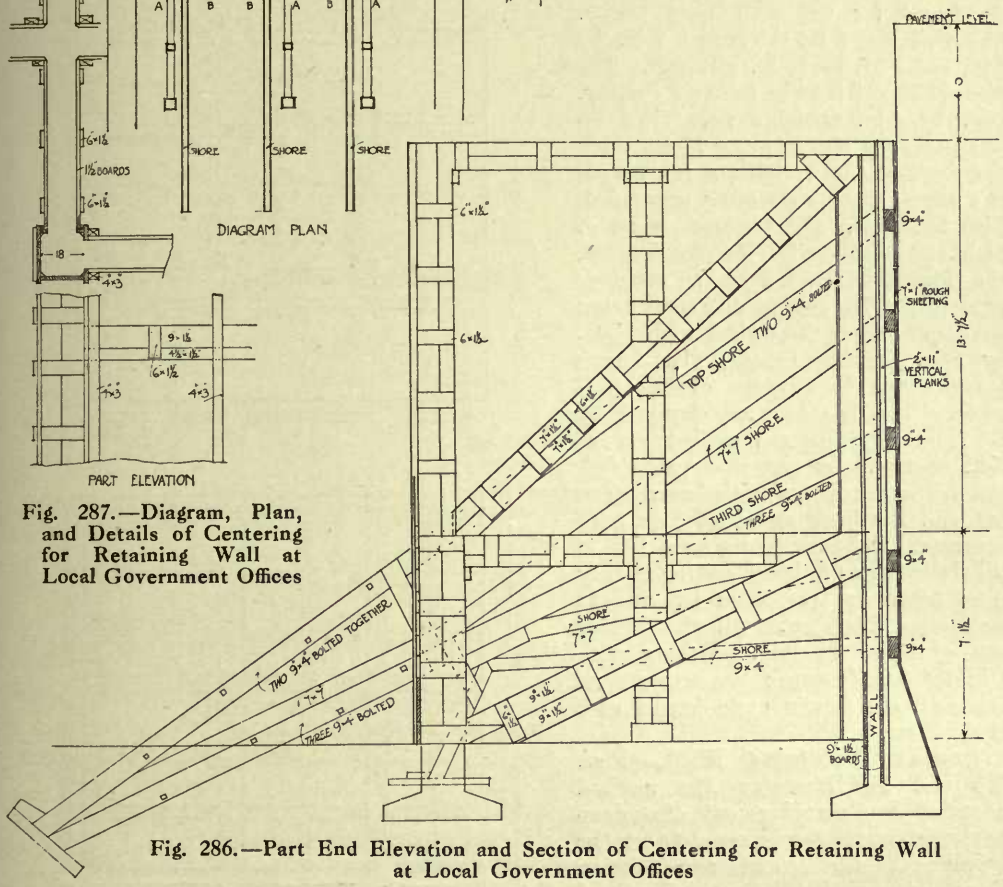


Fig. 286.—Part End Elevation and Section of Centering for Retaining Wall at Local Government Offices

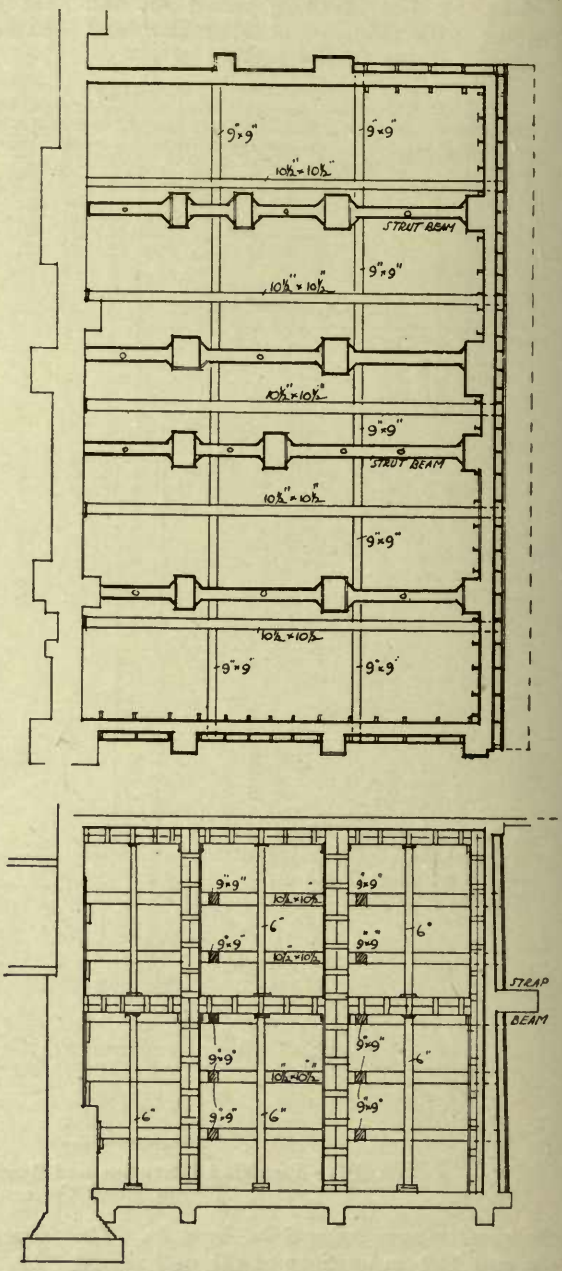
the shorter ones being 2 in. by 4 in. The top and bottom portions of the wall should be strutted independently of each other. The uprights being securely stayed, the first two $1\frac{1}{4}$-in. by 8-in. boards are placed on the wall side of the uprights. The back is then proceeded with similarly.

The two sides of the buttresses are formed Local Government Offices, carried out by the Trussed Concrete Steel Co., are shown by Figs. 286 and 287. The concrete at the basement level is more than 24 ft. below the street level, and the excavation was done in five sections of roughly 4 ft. each. As each section was excavated, 1-in. by 7-in. rough vertical sheeting or poling

boards were placed against the earth, these being secured with 4-in. by 9-in. horizontal wales and 2-in. by 11-in. vertical planks and raking shores. The shores were placed 11 ft. or 12 ft. apart. The top and third shores were formed with 4-in. by 9-in. timbers bolted together, the intermediate shores being 7-in. by 7-in. balks. As will be seen from the illustration, the thrust of the earth against the retaining wall was resisted by trussed and braced buttressed division walls with intermediate pilasters.

When the earth was properly held up, the forms for the wall were begun. The wall was 5 in. thick with 9-in. by 9-in. and 9-in. by 12-in. pilasters. The face of the wall is 14 in. from the cutting. 3-in. by 4-in. uprights were fixed up as shown, the boards being 1½ in. thick and fixed to the uprights a few boards at a time as the concreting proceeded. The forms for the pilasters were of 1½-in. by 9-in. vertical flooring boards, the front being added as the concrete proceeded. The column and strut boxes were framed on three sides with 1¼-in. by 9-in. flooring boards, the fourth being added in advance of the concrete. The boards were clamped by 1-in. by 6-in. battens nailed on at 2-ft. centres. As the wall proceeded, the poling boards and wales were taken out and the back space filled in with concrete. Only the shores were left in till the wall was finished, and when they were taken out the holes left in the wall were filled up. All the boxing was of 1½-in. by 9-in. boarding with 1-in. by 6-in. battens; the arrises were taken off the columns and beams with angle fillets.

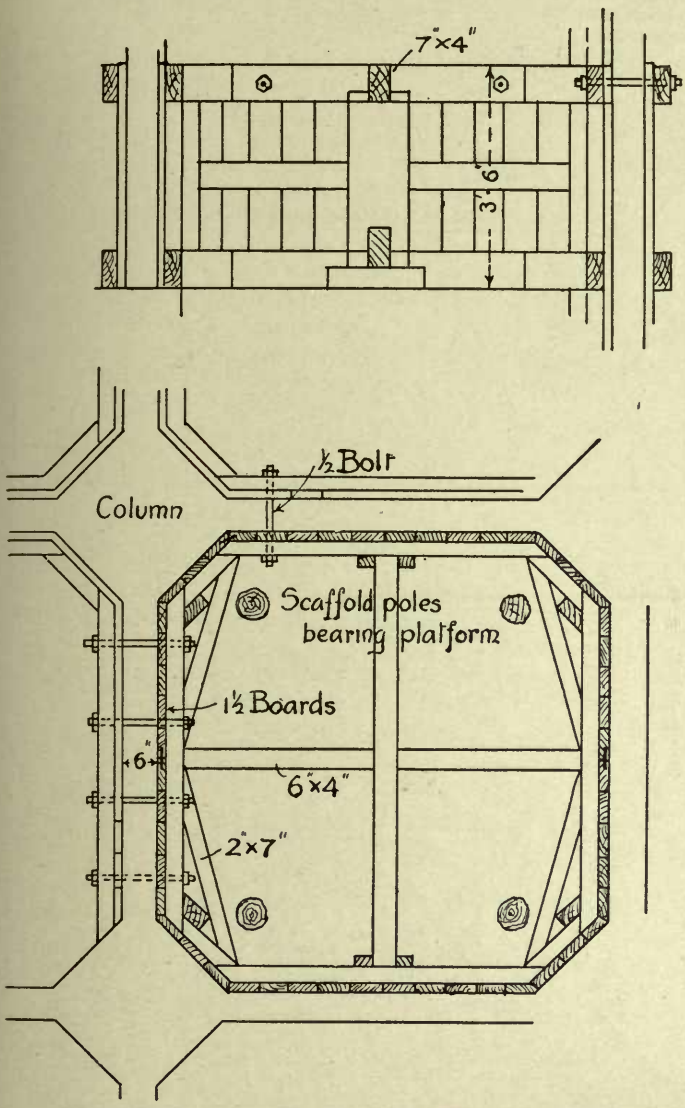
Another Retaining Wall.—Figs. 288 and 289 illustrate the method adopted in carrying out the deep basement in reinforced concrete at the premises of the British and Foreign Blind Association, Great Portland Street, London. The basement has an area of 35 ft. by 55 ft., and is 31 ft. below the level of the pavement. The walls on three sides are reinforced against a possible 24 ft. head of water. The fourth side being an existing brick wall is deep enough to render underpinning unnecessary. The excavation was carried out in six sections of 5 ft. deep. Immediately the first section was excavated, 1½ in. upright sheeting was laid to the earth,



Figs. 288 and 289.—Plan and Section of Centering, etc., for Retaining Wall in Deep Basement

waled in the centre with 10 in. plank, and strutted right across the basement with five balks 10½ in. square and 9 ft. apart; folding wedges were used to tighten up

the struts against the brickwork on the fourth side. Three cross struts, 9 in. by 9 in., were used to stay the sheeting on the end walls, and were continued as stays to the large 10½-in. balks, thus making a rigid frame. The balks and struts had to be placed so as not to hinder the carrying out of work on the pillars. The next section was proceeded with similarly, until the excavation was carried down to the full depth. The wall was 14 in. at the bottom, tapering to 6¼ in. at the top. Half way up the walls, on the outside, large strap beams were introduced to meet the thrust of the earth and water, the thrust being further met by introducing horizontal reinforced concrete strut-beams between the walls and the internal pillars, which carry heavy weights from the superstructure. Horizontally, the wall was strengthened by the addition of pilasters. In order that the wall should be monolithic it was carried up in continuous layers.



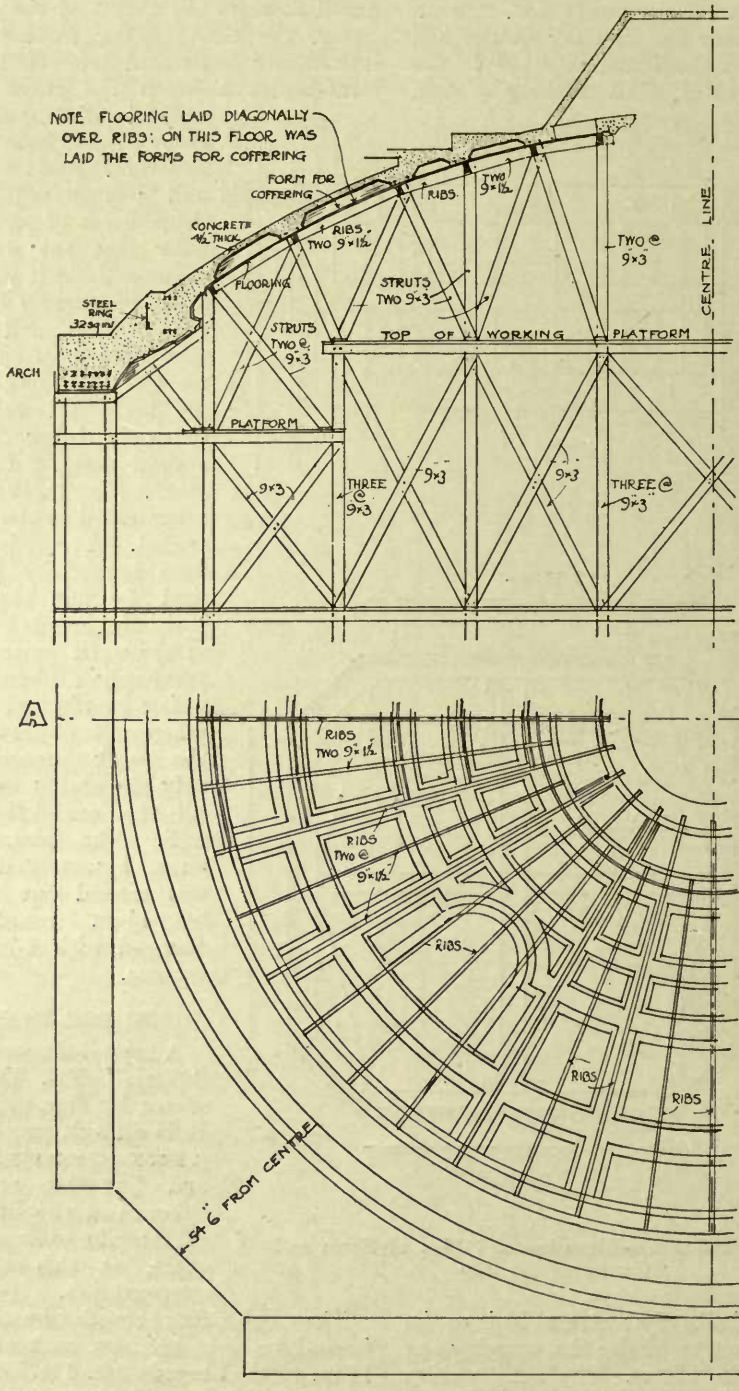
Figs. 290 and 291.—Elevation and Plan of Form and Centering for Silo

The walls and pilasters were carried up as described for the preceding example, the horizontal boarding of the walls being secured by upright posts, 2 ft. to 3 ft. apart, the outer boarding being added as the work proceeded. The pilaster sides were formed by 1½-in. vertical flooring, the front of horizontal boards being added as the work proceeded. As previously described, the poling boards and walls were taken out at the back as the work proceeded, and only the shores were left in till the completion of the wall. The boxing of the strut beams and columns was carried out with 9-in. by 1½-in. boarding, and clamped with 4-in. by 1-in. battens.

FORMS FOR SILOS

A typical silo erected at the "Hovis" Mill, Vauxhall, is shown by Figs. 290 and 291. It is difficult properly to reinforce a rectangular corner, and for this reason the columns at the intersections of the silo walls are on the angle, as this makes the strongest job.

The moulds for the walls of each bin were formed in eight sections, each with planed 1½-in. vertical boards 3 ft. 6 in. high, secured to top and bottom horizontal ledgers 2 in. by 7 in., and a 2-in. by 5-in. middle rail; placed against these were planks rebated to receive the ends of the 4-in. by 7-in.



Figs. 292 and 293.—Vertical Section and Plan of Centering for Dome of Wesleyan Hall, Westminster

horizontal stays by which the forms were kept in position, the proper distance apart, and which in turn were cross-strutted with 4-in. by 6-in. stays. The forms at the angle column were kept in position with split 2-in. by 7-in. vertical and 2-in. by 7-in. angle struts placed horizontally, two in the height of the form. The forms between two adjoining compartments were kept the proper distance apart at the head by long bolts as shown. The concreting was carried up in heights of 2 ft. at a time.

The forms for the hopper at the bottom of the silo consisted of $1\frac{1}{2}$-in. dressed boards on 2-in. by 5-in. raking struts carried down to the platform and cross braced with battens. The upper inside boards were laid as the concreting proceeded.

Forms for circular silos are constructed as described later for circular tanks.

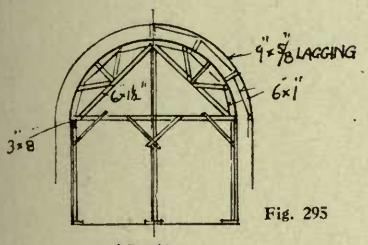


Fig. 295

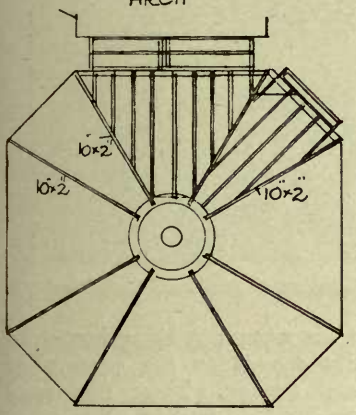


Fig. 296

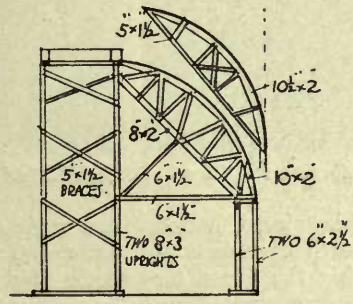


Fig. 297

Figs. 295 to 297.—Centering for Octagonal Dome Circular in Section

FORMS FOR DOMES

The centering requisite for erecting domes is formed of numerous uprights carried up to the springing of the arch from a firm footing below, and consists either of scaffold poles spaced at 8 ft. to 10 ft. centres, and braced with ledgers and cross braces. In other cases, the uprights are built of 3 in. by 9 in. timbers bolted together with horizontal cills and cross braces, similar to a gantry for a derrick.

The art of disposing timber to form a centre capable of supporting the weight of the material placed on it, and ascertaining

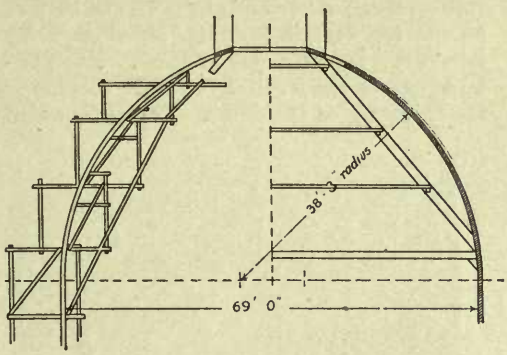


Fig. 294.—Centering for Dome at Annapolis New Academy

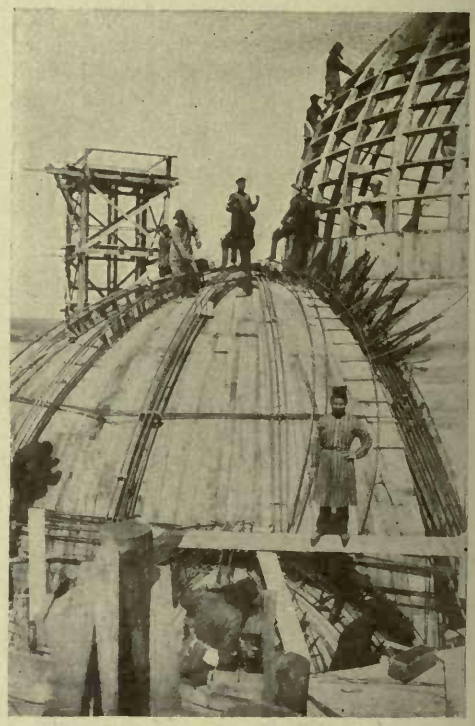
the scantling required, demands greater knowledge than the ordinary builder usually possesses; but fortunately, in Great Britain, at any rate, centres usually err on the side of safety.

The following examples will give a fair idea of the nature of dome centering commonly met with in practice.

Dome of Westminster Hall.—At the Wesleyan Hall, Westminster, is probably the finest example of a reinforced concrete dome to be found in Great Britain, its span being 109 ft. over the pendentives. The shell of the dome is only $4\frac{1}{2}$ in. thick, strengthened by 16 vertical ribs and 7 horizontal ribs or rings, three of which latter are reinforced to take the thrust. For this the bottom main rib is reinforced with an area of 32 in. of steel. These cross ribs form an ornamental coffering which is very effective. The centering requisite for building this dome was carried on a platform which was built up from the concrete floor of the hall. Planks were laid on the floor, and at from 8 ft. to 10 ft. centres uprights consisting of

three 3-in. by 9-in. timbers bolted together were erected, together with horizontal ledgers and cross braces. On the top of these standards were laid ledgers which supported the working platform of heavy planks. Between the central uprights in the middle of the hall, the lift was erected, this carrying material right from the bottom to the top of the building, openings being left in the floor for this purpose. The platform was at the level of the main cornice at the springing of the dome. The ribs, which consist of two 1½-in. by 9-in. pieces, bolted together, were entirely floored over diagonally, this flooring supporting the forms for the coffering slabs and beams, treated in every respect similarly to floors. These ribs were supported by struts and uprights consisting of two 3-in. by 9-in. timbers bolted together. Lagging was only erected to support the underside of the slab. The upper face of the slab concrete was worked up from the springing by trowel work. The centering is clearly shown in Figs. 292 and 293.

Dome of "Morning Post" Building, London.—This was constructed on an eight-rib rolled steel joist framework. The filling in was on the Columbian system of reinforced concrete by cruciform bars suspended to the ribs by stirrups. An internal and external shuttering was used. The inner shuttering was nailed to the horizontal radiating ribs placed 1 ft. 6 in. to 2 ft. apart, resting on triangular brackets fixed to 3-in. by 7-in. uprights and raking struts, the bottom of the latter being secured to planks laid on the floor. A portion of the external shuttering sufficient for a part of the dome was made on the ground, it being formed of horizontal ribs shaped to the contour of both plan and section. These ribs were braced by 3-in. by 7-in. boards. The dome was concreted in sections between the steel ribs, the concrete being rammed from the top, the same portion of shuttering being used for this purpose. When the lower portions were concreted the shuttering was raised by wire guys.

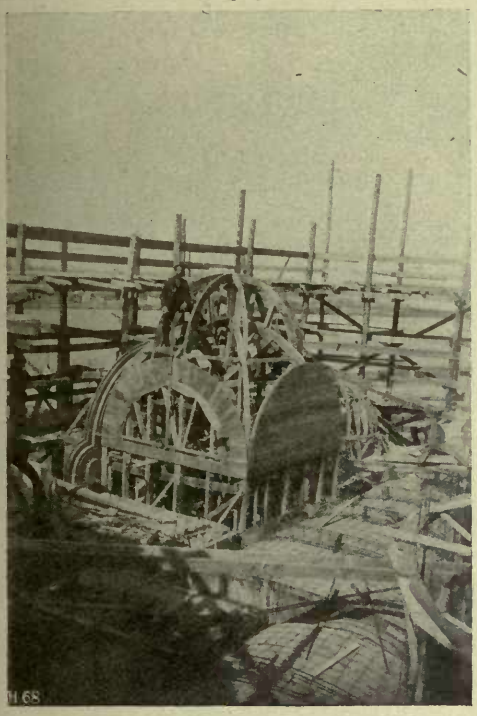


Figs. 298 and 299.—Reinforcing the Large and Small Half Cupolas of the Poti Cathedral

A 20-ft. Saucer Dome.—The forms for a saucer dome 20 ft. in diameter, supported on two sides of a barrel vault, which was carried out at the chapel of St. Charles College, Notting Hill, London, are very

interesting. A platform was erected at the level of the springing of the arch. From this platform, resting on the planks, were raised 3-in. by 7-in. uprights wedged up to support at the base of the saucer dome a circular ring or plate formed of three thicknesses of $1\frac{1}{2}$-in. boarding, which in turn supported the bottom ends of 3-in. by 9-in. radiating ribs shaped to the contour of the dome. On these ribs were nailed close-boarded lagging. The arches to the barrel vaults were supported on spliced short ribs measuring 3 in. by 7 in., supported at the joints by 2-in. by 5-in. struts, the ends of which rested on a 3-in. by 7-in. tie wedged up from the platform. The pendentives were filled in with 3-in. by 9-in. shaped ribs.

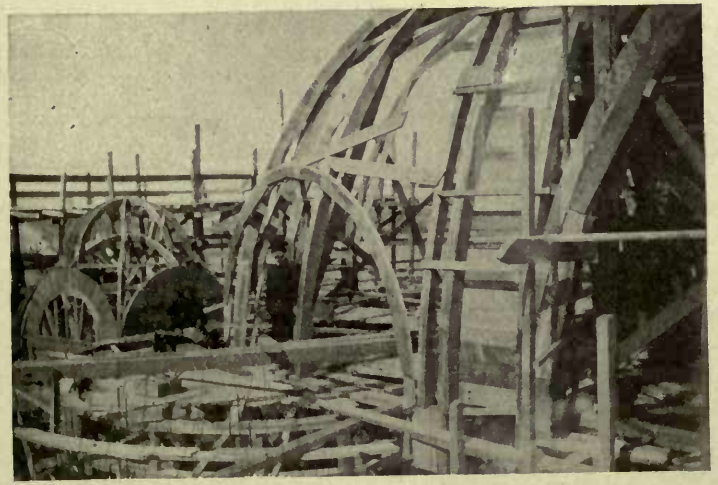
Dome of Annapolis New Academy.—The dome at the new academy at Annapolis, U.S.A., shown by Fig. 294, is a wonderful piece of construction. Its peculiarity is that no permanent centering was used, the mould and the supporting frame being built upwards as the work progressed, receiving its support from the work already done. The forms were kept on the concrete only long enough for it to harden, then raised 2 ft. and clamped in the new position. The moulds for the outer form were first put in place, and the inner forms were hung from them. This was carried up gradually until the men were practically working on a floor before the crown was reached. The base of the dome, which was 69 ft. in diameter, overhung the main walls below about 7 ft. all round, the diameter of the main building being 83 ft. Thus heavy corbelling was necessitated to carry the bottom ring of the dome.

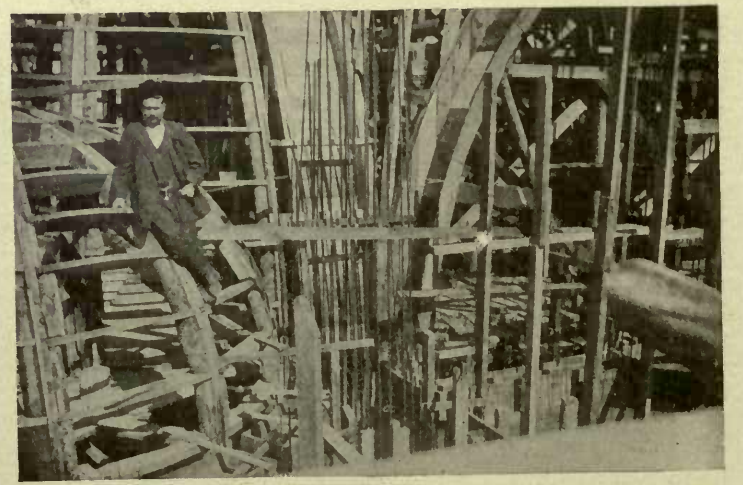


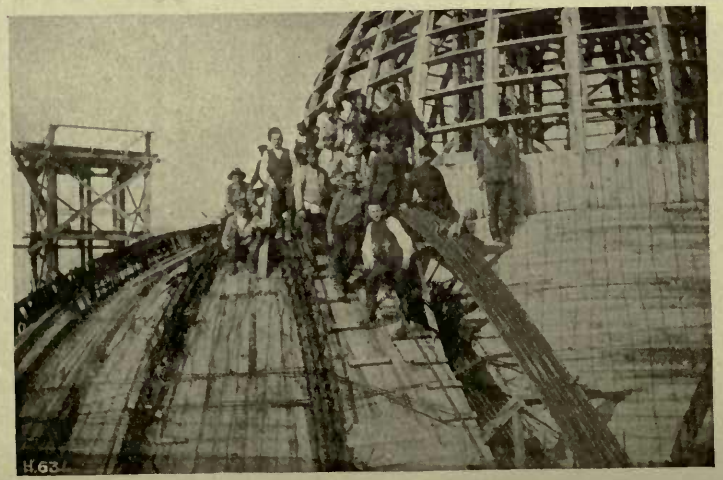
Figs. 300 and 301.—Centerings for Belfry and Arches of the Poti Cathedral

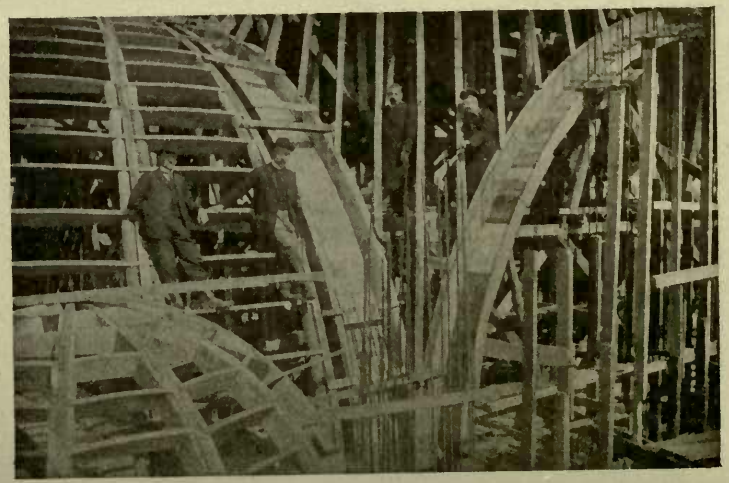
The cathedral at Poti, Russia, is described in a later chapter, but it is convenient to give here a number of photographs showing the construction of the dome (*see* Figs. 298 to 305).

Octagonal Dome.—Figs. 295 to 297 show the centering used for an octagonal dome, which is circular in section, at a mortuary chapel in New York. Fig. 296 includes a part-plan of the ribs. The main section is through the angle ribs, a section through the common ribs being shown adjoining. The main dimensions of the centering are figured in the drawings.









Figs. 302 to 305.—Centerings and Reinforcement for Cupolas and Vaulting at the Poti Cathedral

FORMS FOR TALL CHIMNEYS

The most economical material for chimney shafts of circular section is reinforced concrete, and the simplest forms for shaft construction consist of two sets of inner and outer moulds, each 3 ft. high (*see* Fig. 306), held together by means of latches which can be readily undone to enable the forms to be taken away easily. When the concrete is filled in to the top of the upper ring the bottom set of moulds is released and placed

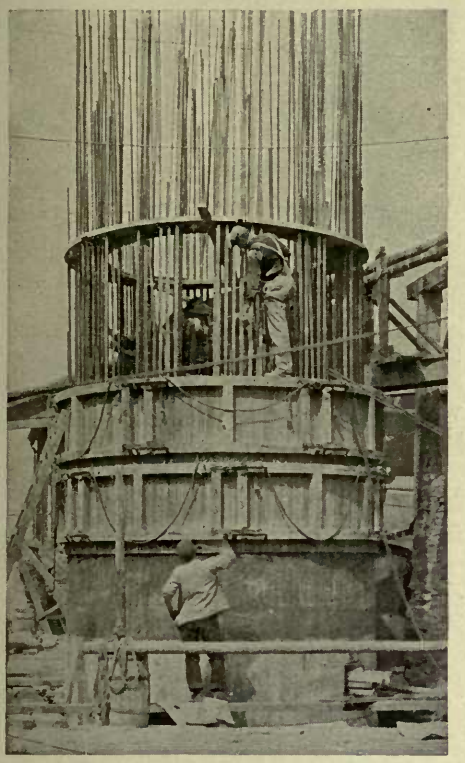


Fig. 306.—Centering, etc., for Chimney at Northfleet

on the top of the upper moulds, left in position and safely held there by the frictional resistance of the concrete; this course is repeated section by section till the whole is complete. A circular wooden gauge, made of two $\frac{3}{4}$-in. layers, is placed 6 ft. above the level of the top form to hold the rods in true alignment, and it is raised as the work proceeds.

As the work is carried up from the inside, only a light scaffold is required, built up section by section every 4 ft. or 5 ft., the framing of each section consisting of four uprights to support a square platform of stout planks, holed in the centre for hoisting the materials by bucket. Double-ring chimneys are carried up at the rate of one form, and single-ring chimneys at the rate of two forms a day.

The Ransome System.—The moulds and false-work used by the Ransome Company in the United States for chimney shafts are much more complicated. A square scaffold tower is raised in the centre of the chimney with four 4-in. by 6-in. corner posts, and 1-in. by 6-in. cross braces with 2-in. by 10-in. horizontal braces every 4 ft. or 5 ft., and built up a few feet in advance of the construction; the scaffold tower supports a platform of horizontal planks. From the tower project beams, and from these the inner and outer moulds are suspended by means of four vertical rods having their upper ends threaded and engaging in screw wheel bearings so that the lower moulds can be raised or lowered. On the top of the form mould on the outside, light brackets strutted out also support a working platform. To avoid removing the cross beams at the head for each set of moulds, telescoping scaffolding is built inside. Inside the chimney a staging is suspended from the tower from which the workmen place and tamp the concrete. The shell moulds are 12 ft. high. The six-sided lower platform is for the workmen who finish the outer surface of the concrete.

FORMS AND CENTERING FOR ARCHES AND BRIDGES

In designing and building forms and centering for arches, care must be taken that the centering is framed up strongly enough to take the weight to come upon it; that, while the material is used economically, the work must be capable of being easily taken down; that the footing is strong enough to support the verticals without settlement when fully weighted; that the lagging round the centering is strong enough to take the weight of the arch ring without deflection; that the centering is properly held up in position by folding wedges, these allowing the centering to be lowered very gradually so that the safety of the executed work can be tested before finally striking.

In general, the forms for arches for reinforced concrete are substantially the same as for masonry and brickwork, and present no difficulty to anyone acquainted with the latter. The lagging round the centering

should be dressed to a uniform thickness, and to a smooth surface.

Centerings for Flat Bridges.—Concrete bridges may be classified under two heads, namely, flat bridges and arch bridges. Flat bridges are either straight flat slabs or combined slabs and girders, and are adaptable for spans up to 40 ft. The centering for a bridge of this latter type is shown by Fig. 307, and, as it bears a strong resemblance to beam and floor forms already described, it needs no further explanation.

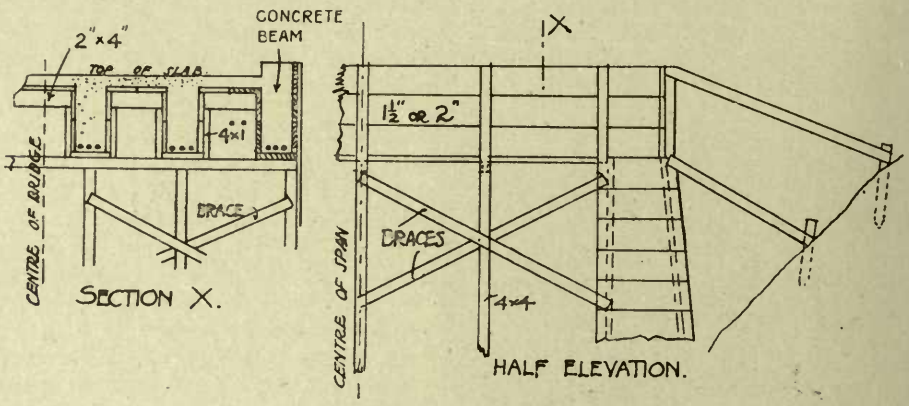


Fig. 307.—Centering for Flat Bridge

Centerings for Arched Bridges.—Arched bridges are constructed with or without ribs. If the arches are reinforced with steel joists or lattice girders, they are known as Melan Arches, and the centering is partly hung from the reinforcement, as shown in Fig. 308.

Arched Roof Principal at Hammersmith Baths.—Fig. 309 shows the forms for an arched principal in reinforced concrete at Hammersmith Baths, London, one of the first of the kind to be carried out in Great Britain. In accordance with the usual practice, a working platform was carried up to the springing. The ribs supporting the forms were out of 2-in. by 7-in. stuff, in short lengths supported at the junctions by raking and vertical struts braced horizontally with 2-in. by 7-in. ledgers. On the ribs were laid 2-in. by 4-in. lagging, on which was nailed 1-in. boarding bent to the contour and forming the bottom. The sides were formed of 1½-in. boarding clamped with 1-in. by 3-in. battens, and were supported on the 2-in. by 4-in. projecting lagging. The sides were battened also on top to keep the forms the proper distance apart. The false-work is well shown in the photograph (Fig. 310).

Centerings for Arch Ring Bridge.—The centering for a 40-ft. span arch ring bridge is shown by Figs. 311 and 312. 8-in. by 8-in. piles are driven in, sawn off near the top, transverse runners are bolted to the piles, and large folding wedges are placed between the tops and the transverse 10-in. by 10-in. beams, which support the weight of the centering above. The centering consists of a set of caps or transverse beams resting on the wedges above the

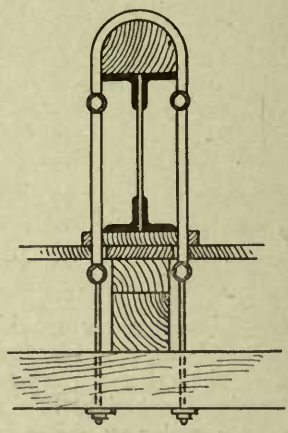


Fig. 308.—Centering for Melan Arched Bridge

pile caps, some shaped bearers notched on and supported by the upper transome beams, and finally of a closely laid lagging resting on the bearers. The last-named are of varying size, depending upon the distance between the verticals and the weight to be carried. For arches having spans up to

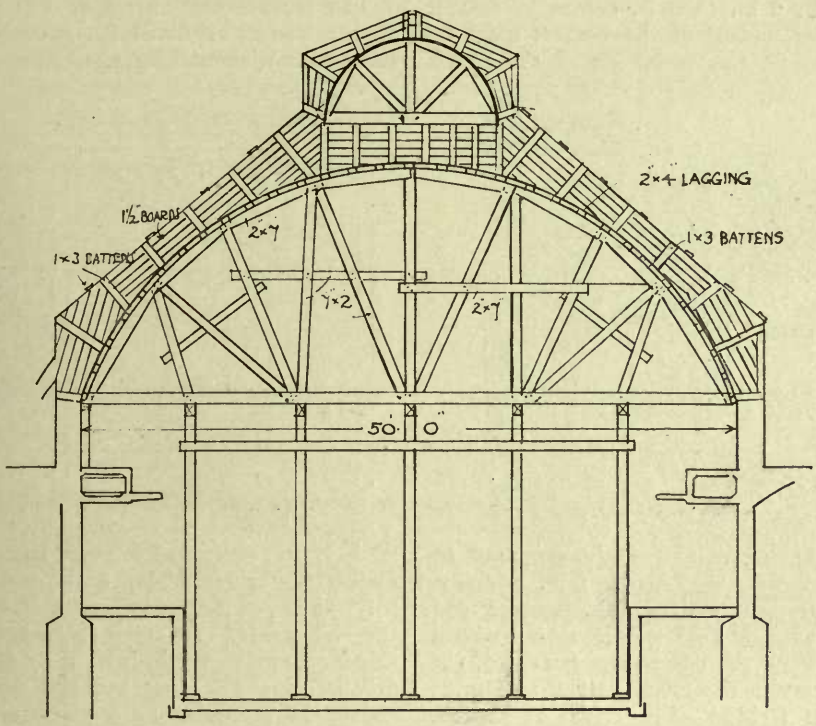


Fig. 309.—Elevation of Centering for Arched Principal, Hammersmith Baths

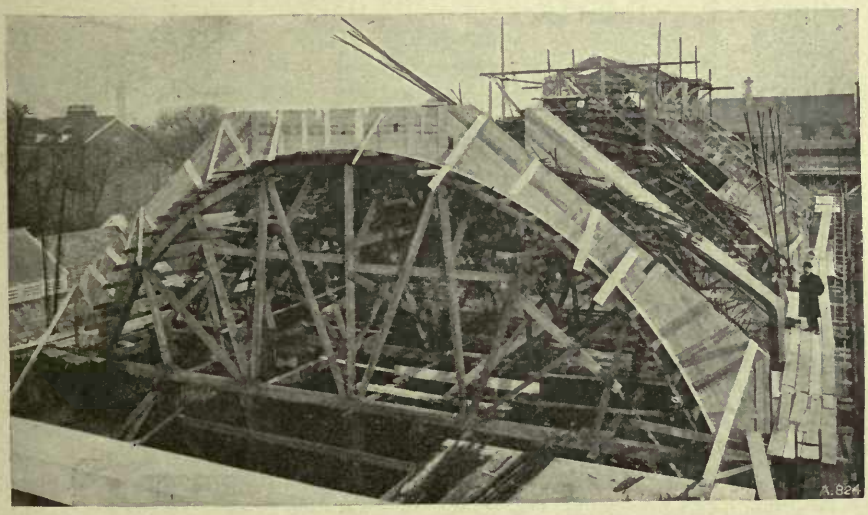
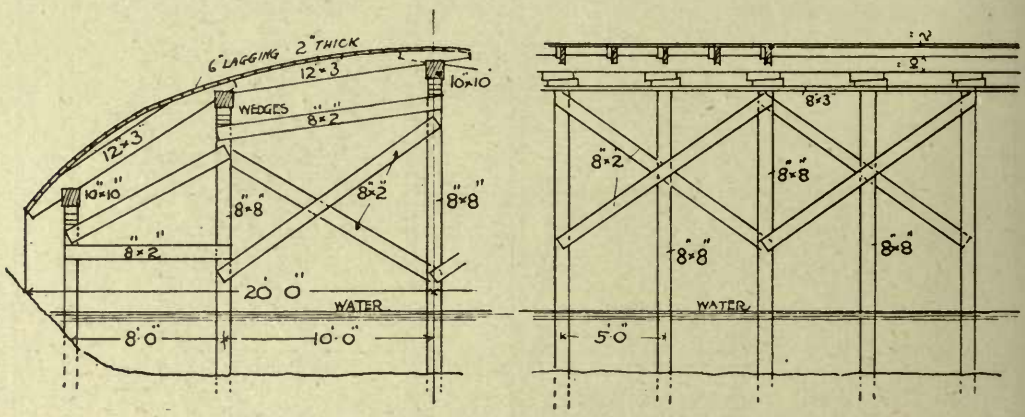


Fig. 310.—View of Centering for Arched Principal, Hammersmith Baths

100 ft. these bearers are from 2 in. to 4 in. wide and 12 in. to 14 in. deep, and spaced from 1 ft. 6 in. to 3 ft. centre to centre. The upper surface of the bearers must be curved to fit the curvature of the under surface of the arch, and the bearers must be braced laterally by 1-in. by 6-in. bridging. The lagging consists of $\frac{7}{8}$-in. tongued and grooved pine or 2-in. spruce with bevelled edges. When the bearers are far apart, the lagging may have to be as thick as 4 in.



Figs. 311 and 312.—Centering for Arch Ring Bridge

Teufen Bridge.—The bridge at Teufen, Switzerland (*see* the photographs, Figs. 315 and 507) designed by such an undoubted authority as Prof. Mörsch, presents points of great interest. The centering, Fig. 313, designed by the professor, is a model to be safely followed in similar situations, as the permanent deflection at the completion of the work, when the centering was removed, was found to be only $\frac{3}{16}$ in.

The main arch, of about 259 ft. span with a rise of about 87 ft., is designed for fixed ends without hinges. The roadway is 22 ft. 7 in. wide, and is more than 216 ft. above the river. The arch voussoir is 3 ft. 11 in. at the crown and 6 ft. 11 in. at the abutments. To carry the centering for this arch, twelve verticals six rows deep were driven below the river bed and carried up above the springing as a platform. Each vertical or standard was formed of two 12-in. by 12-in. balks bolted together and stayed with ledgers and cross braces, both longitudinally and transversely, at every 29 ft. of height. On top of these standards were placed 9-in. by 12-in. ties cleated with angle irons on the under-side to the

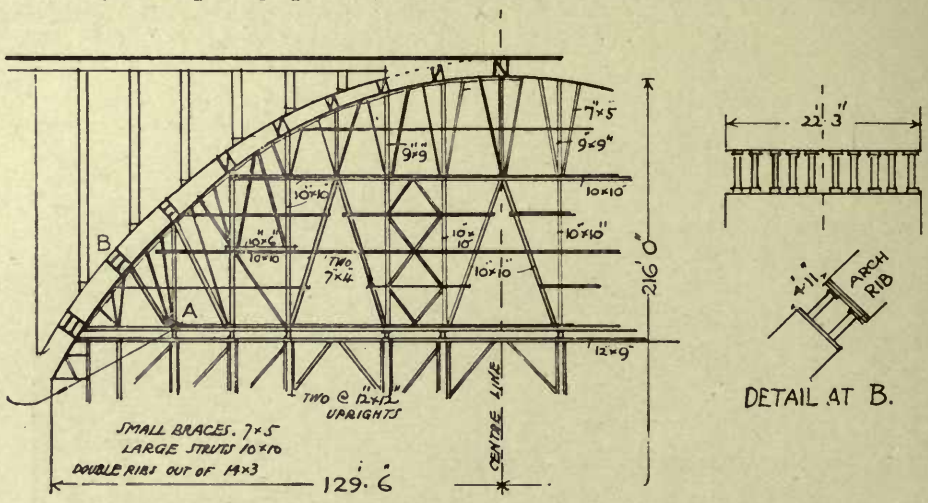


Fig. 313.—Part Elevation and Detail of Centering for Bridge at Teufen

verticals; a 12-in. by 4-in. by 40-in. sole-piece was bolted to the ties, on which was laid two sand boxes, except on the end row adjoining the abutments. A similar sole-plate rests on the sand boxes, and supports the tie beam of the arch centering proper. Pieces of channel iron were employed to

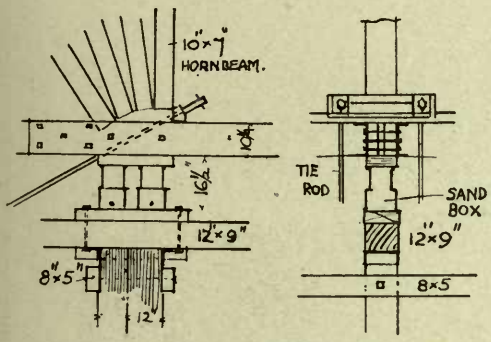


Fig. 314.—Detail of Bridge Centering (see A, Fig. 313)

distribute the pressure of the vertical and horizontal timbers on to the sole-plates and cross beams of the scaffolding direct on to the verticals, and special steel stirrup ties held in the feet of the struts, as shown in Fig. 314. The main struts were 10-in. by 10-in., and the secondary and small struts 9 in. by 9 in. and 5 in. by 7 in. respectively.

from 4 ft. to 5 ft. wide and the whole width of the bridge were left unconcreted; these

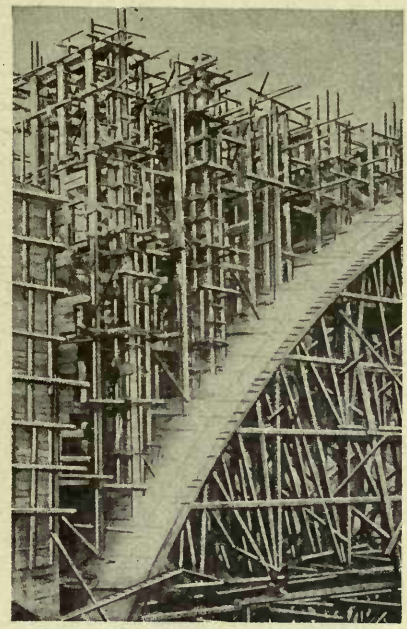
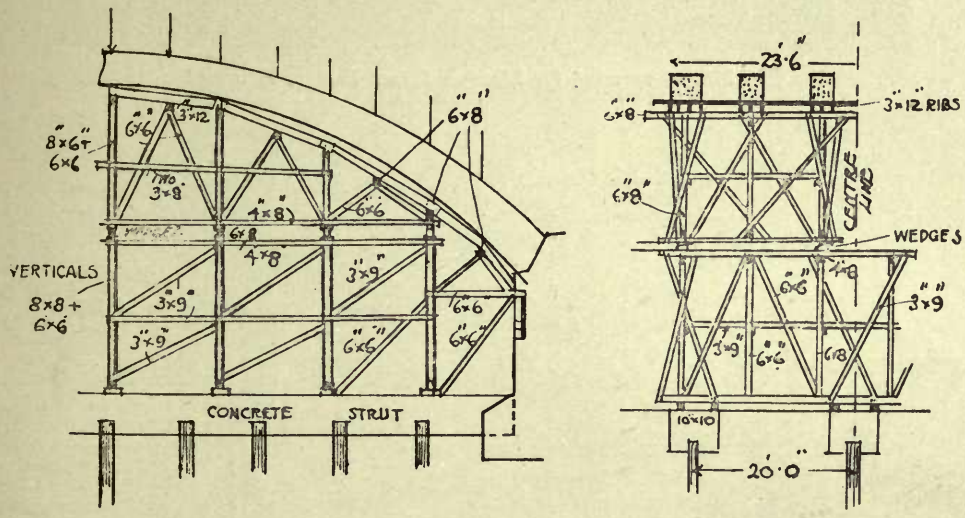


Fig. 315.—View of Part of False-work for Bridge at Teufen

sections of the ring were strutted apart by heavy balk timbers, as shown. After the



Figs. 316 and 317.—Centering for Almandares Bridge, Havana

The struts supported the shaped stringers and laggings, each stringer being composed of two 13½-in. by 3¼-in. planks. The arch ring was concreted in sections, and portions

ring had taken its proper bearings, the concreting of the omitted parts of the arch ring was completed.

Arch bridges formed of a series of arched

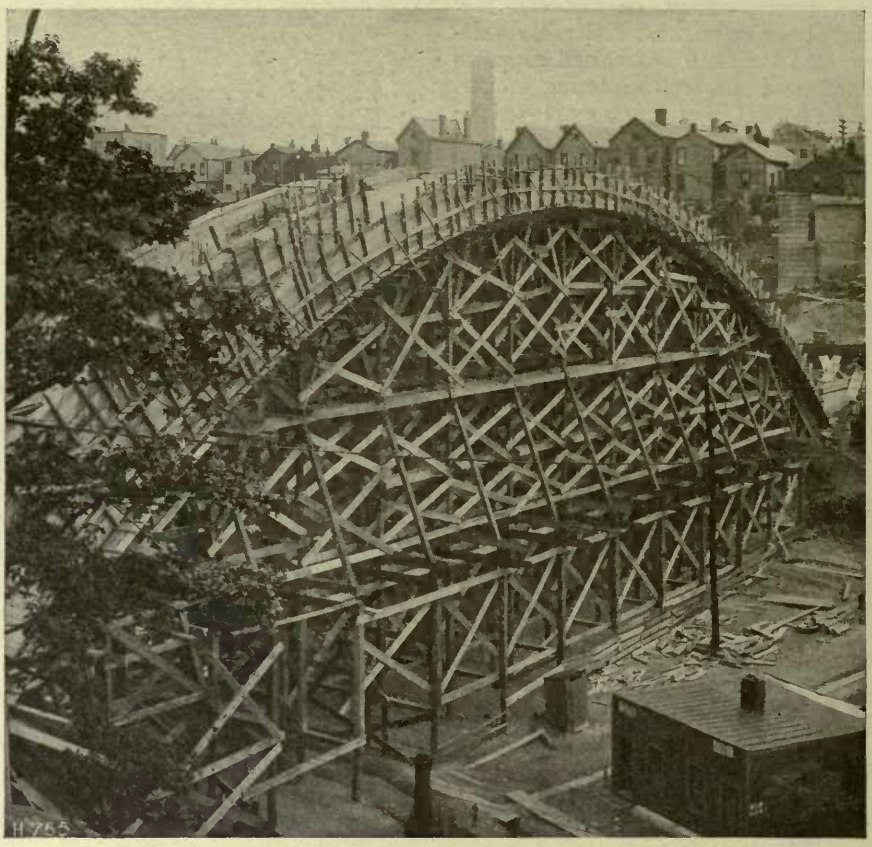
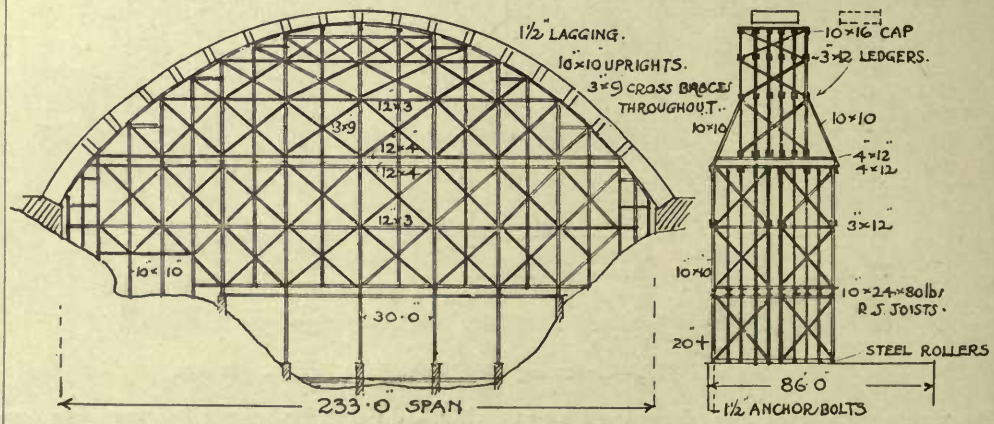


Fig. 318.—Centering for Meadow Street Bridge, Pittsburg



Figs. 319 and 320.—Centering for Bridge of 233-ft. span

Fig. 321.—Centering for Bridge over River Werra in Thuringia

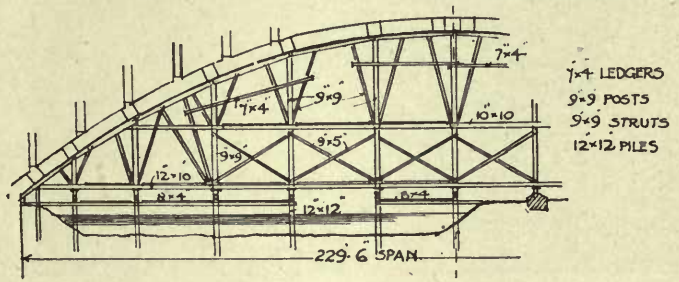


Fig. 322.—Centering for Bridge of 229-ft. 6-in. span

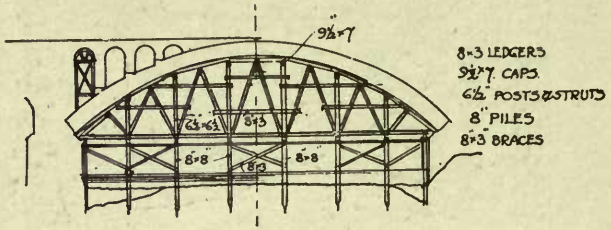


Fig. 323.—Centering for Small Bridge

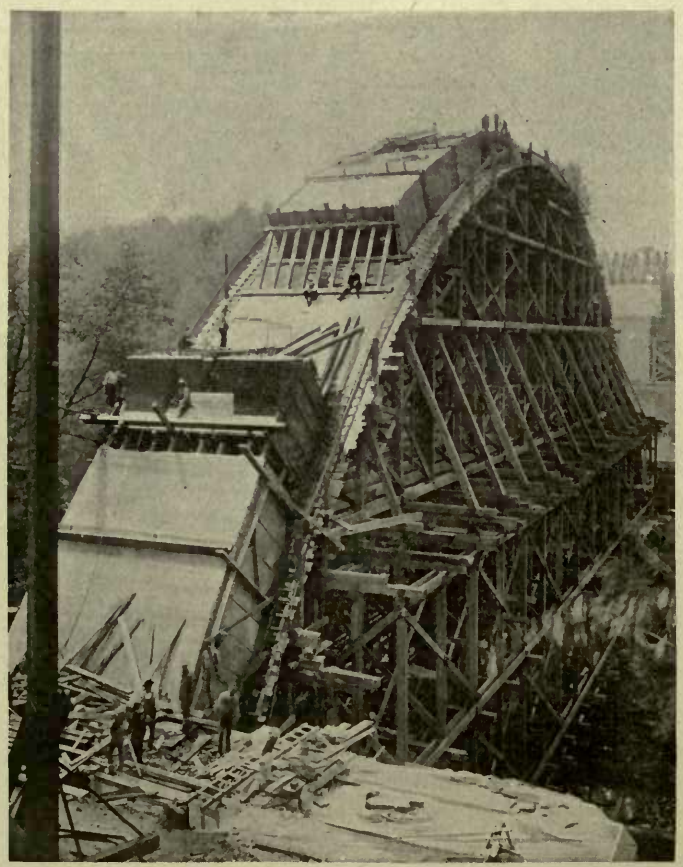


Fig. 324.—Centering for Walnut Lane Bridge, Philadelphia

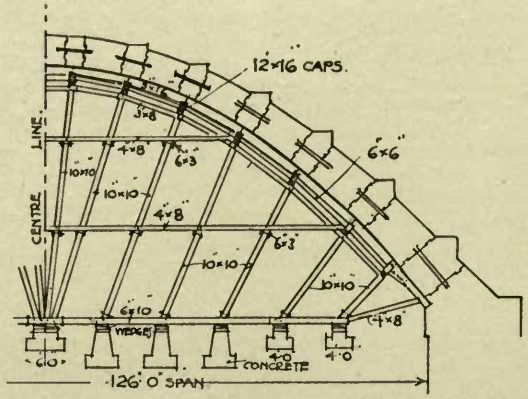


Fig. 325.—Centering for Bridge of 126-ft. span

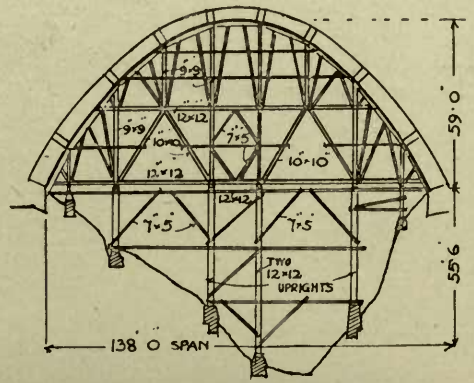


Fig. 326.—Centering for Bridge of 138-ft. span

ribs supporting slabs have the ribs generally first erected on such centering as has been described for the Teufen Bridge. For completing the slabs after the ribs have hardened

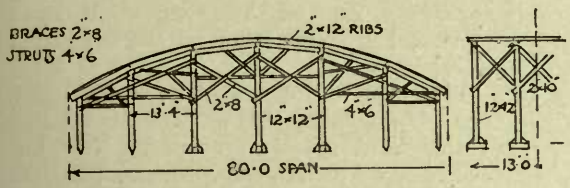


Fig. 327.—Centering for Bridge of 80-ft. span

somewhat, false-work similar to that described for floors is used.

Seventeen examples of bridge centering culled from the best Continental and American practice are shown in outline or in photographic view by Figs. 316 to 332, the sizes of the main timbers being given as far as obtainable.

SUSPENDED CENTERING FOR BRIDGES

In the building of bridges over running streams great difficulties have been experienced owing to the difficulty of erecting the centering and owing to the serious settlements of the centering that frequently take place. American engineers found that the running water washed round about the base of the wood piles and destroyed their vertical friction, in one case causing a settlement of 12 in. Disaster would have occurred had not the arch been constructed in sections, as at Teufen, and the voussoirs between these sections not yet concreted in. The voussoirs were raised to their true position by screw-jacks, which had to be continually adjusted to counteract the sinking still going on. This led to the introduction of flexible suspended centering, and the particular design about to be described was used in Chickahominy River Bridge, Richmond, Va., U.S.A. Fig. 333 shows cables hung over the piers and anchored at the abutment of the arches. Attached to these cables are the hangers which support the centering shown in Figs. 334 and 335. In order to pass the cables through the piers or abutments to anchorage, a small pipe or box with proper slope is embedded in the abutments when constructing them.

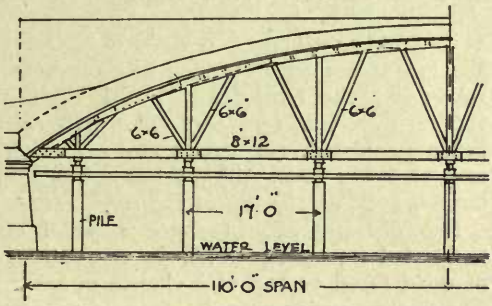
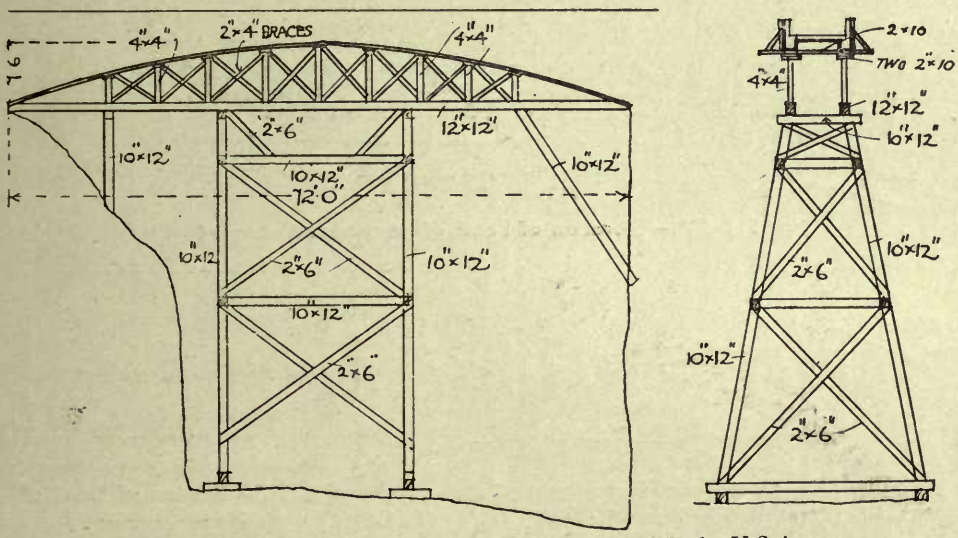


Fig. 328.—Centering for Bridge of 110-ft. span



Figs. 329 and 330.—Centering for Bridge at Deer Park, U.S.A.

The arch can be raised or lowered to the proper curvature by lengthening or shortening the hangers, by means of turnbuckles, or by differential pulleys for very heavy loads.

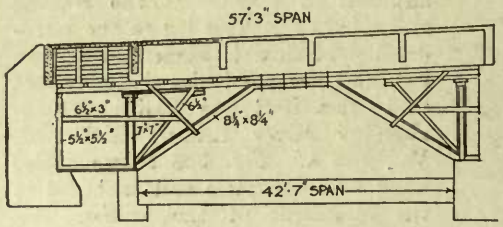


Fig. 331.—Centering for Flat Bridge of 42-ft. 7-in. span

It is usual to set up the arch ribs in sections. The key spaces are then filled with concrete, embedding the reinforcement and binding the arch sections together. The arch ribs being completed, the cables and centering may be taken down and any superstructure desired built on the top of the rib.

Figs. 334 and 335 illustrate a panel of suspended centering which is constructed of wood firred on top with shaped member. The firring member can be changed to suit n arch of different radius. The longitudinal members are 2-in. by 12-in. planks, overlapped at the crossbars and notched to receive them. The cross bars are 3-in. gas barrel, 4 ft. long.

A similar style of construction has been adopted in Ireland for a foot bridge 172 ft. long, and 150 ft. above the level of the sea. The ribs were made in four portions on the shore, and supported by overhead cables in position as before described till the key spaces were filled in. The floor was shuttered in situ, being supported on the completed ribs.

Figs. 336 to 338 are photographs which further illustrate the method of building a suspended flexible centering. Near the abutments are frames or towers, from the top of which hang cables anchored in any suitable way. Wire rope hangers, depending from the cables, carry cross members of wood, iron or steel, upon which rest longitudinal wood or steel shapes so attached that the whole centering is flexible, and any curve may be imparted to it merely by varying the lengths of the hangers; for the finer adjustments, the hangers have turnbuckles. The

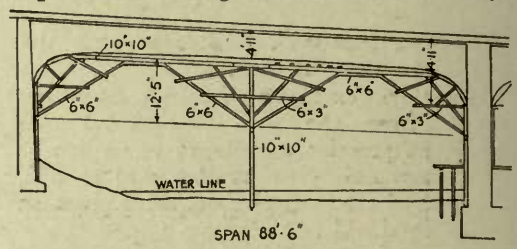


Fig. 332.—Centering for Flat Bridge of 88-ft. 6-in. span

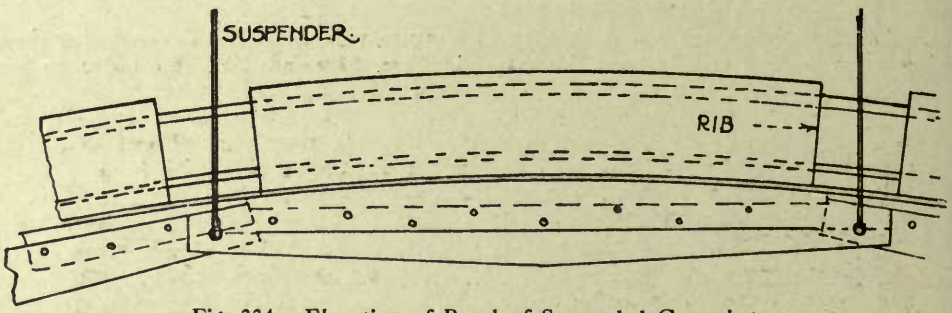


Fig. 334.—Elevation of Panel of Suspended Centering

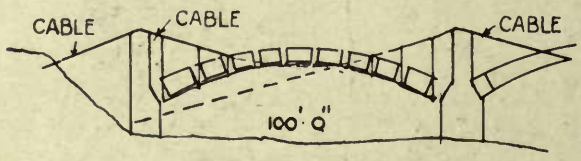


Fig. 333.—One of the Three Spans of the Chickahominy River Bridge

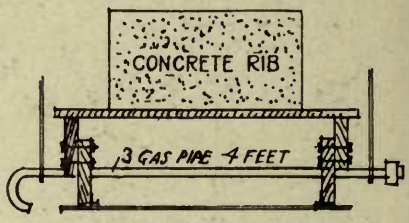


Fig. 335.—Section of Panel of Suspended Centering

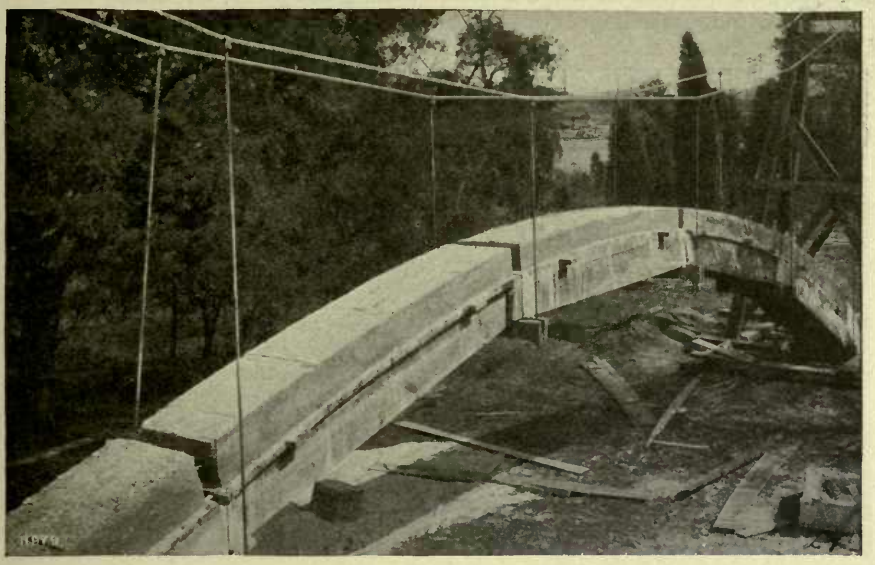


Fig. 336.—Cables supporting Concrete Sections, showing Key Spaces to be filled in

Fig. 337.—Arch Ribs before Striking Suspended Centering

end sections preferably rest on the abutments, the bearings being hinged. The previously moulded voussoir sections of the concrete arch ribs, or, instead, the moulds themselves, may next be laid in place, being supported by the side pieces. The approximate lengths of the hangers is determined in advance, the final adjustments being made by means of the turnbuckles or, in the case of very heavy work, by means of differential pulleys. Targets hang from the cross bars on supports of different lengths, and when they all are at the same level, it is known that the flexible centering has been brought to its proper form. An article by Philip Aylett, in *Concrete* (Vol. VII., pp. 24–35), should be consulted for further particulars, its author being the designer of the examples of flexible centering here illustrated.

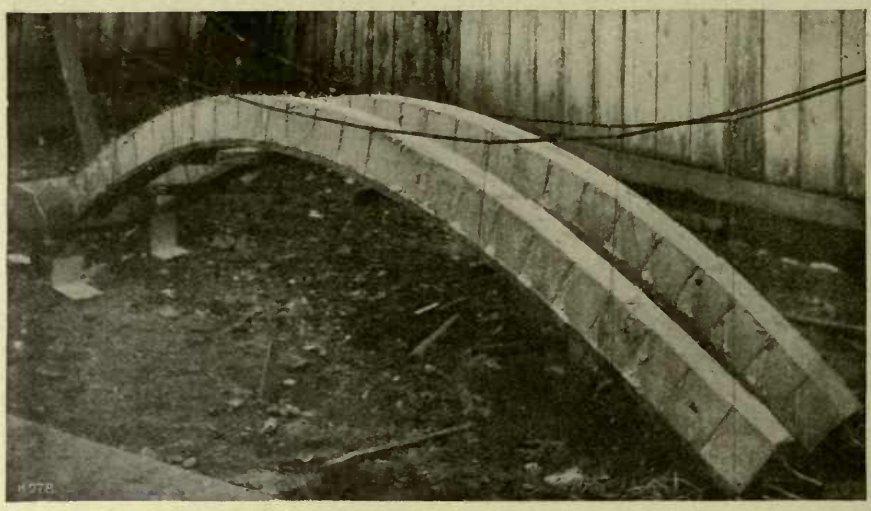


Fig. 338.—Arch Ribs Formed in Suspended Centering

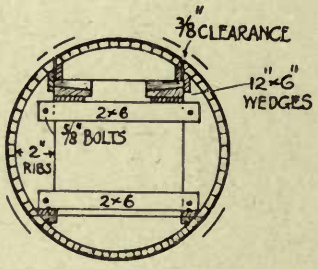


Fig. 339.—Centering for 6-ft. Sewer

FORMS FOR PIPES AND SEWERS

The larger concrete sewers moulded in place are practically monolithic, while the smaller ones are constructed by joining short lengths of concrete pipes together and sealing the joints, so making one continuous pipe. It is not good practice to mould pipes of less than 3 ft. diameter in situ owing to the difficulty of devising suitable forms. When more than 3 ft. in diameter, the pipes can be easily moulded into shapes which have more stability and efficiency than have those of circular cross section. By giving concrete pipes a broad, flat base, they have a better bearing on the foundation.

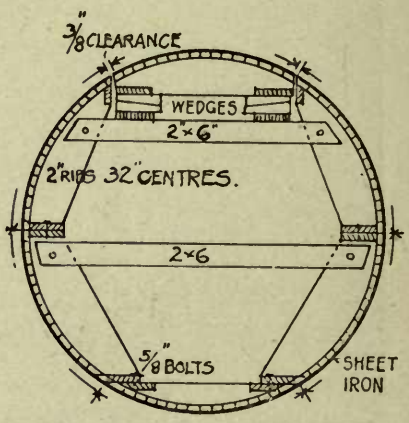


Fig. 340.—Centering for 8-ft. 6-in. Sewer

Fig. 339 shows the centering for a sewer 6 ft. in diameter, constructed in 8 ft. lengths, 16 ft. being done at one operation. To facilitate striking the forms they are

constructed in four portions. The ribs are 2 in. thick, secured to each other by $\frac{5}{8}$-in. bolts at 32-in. centres, and braced by 2-in. by 6-in. bracing secured to the $\frac{5}{8}$-in. bolts. The top section is secured in position by 12-in. by 6-in. wedges. There is $\frac{3}{8}$ in. clearance between the top section and the side section for dropping the centre. 2-in. by 2-in. laggings are fixed to the ribs and covered with No. 27 gauge sheet iron.

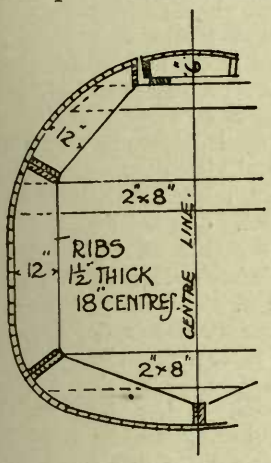


Fig. 341.—Centering for Conduit at Jersey City

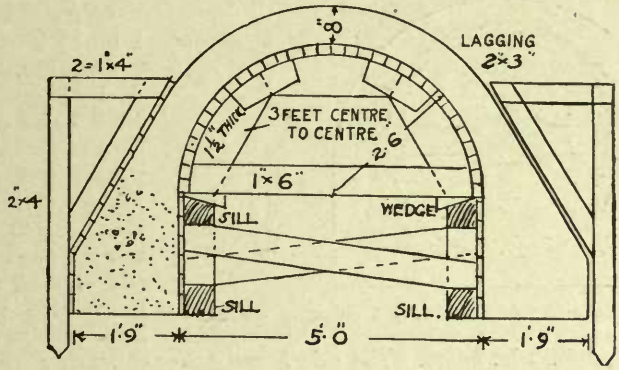


Fig. 342.—Centering for 5-ft. Arched Culvert

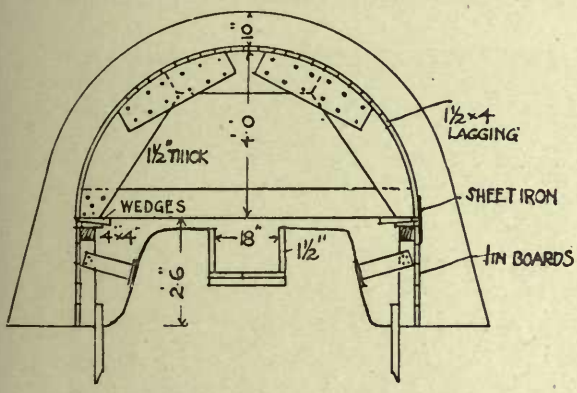


Fig. 343.—Centering for 8-ft. Arched Culvert

Fig. 340 shows the form for a sewer 8 ft. 6 in. wide. This pipe was constructed in six sections, and made in 7-ft. lengths.

Fig. 341 shows the centering for the conduit for water supply for Jersey City, which is constructed in seven sections as shown. The 12-in. by $1\frac{1}{2}$-in. ribs at 18-in. centres are secured by 2-in. by 8-in. braces.

Fig. 342 shows the form for an arched culvert 5 ft. wide. After the soil has been prepared for the concrete, the 4-in. by 4-in. cills should be set and braced with 1-in. by 6-in. braces. The inner arch form is then wedged up from the cills. The ribs are shaped out of $1\frac{1}{2}$-in. stuff and placed 3 ft. apart centre to centre, this necessitating thick lagging (2 in. by 3 in.); 1-in. stuff would do if the ribs were only 2 ft. apart. On the outer side, 1-in. by 4-in. or 1-in. by 6-in. horizontal boards are laid to 2-in. by 4-in. braces and uprights set to the splay of the extrados, as shown.

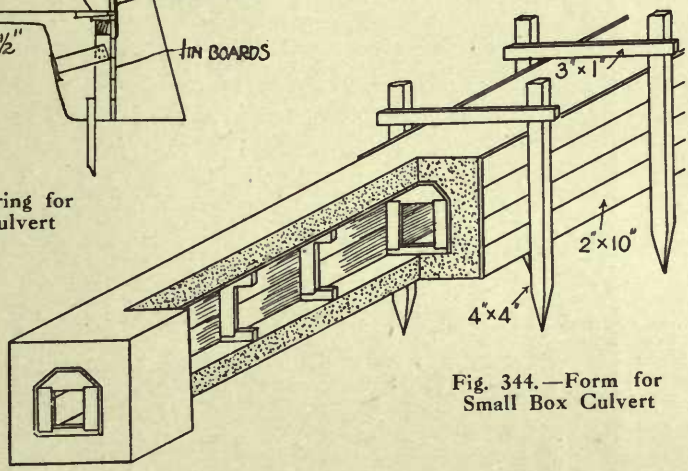


Fig. 344.—Form for Small Box Culvert

Fig. 343 shows the form for an 8-ft. arched culvert. The 1-in. boards of the wall form are secured by 2-in. by 4-in. verticals staked

into the ground and strutted against dumping left in; a 4-in. by 4-in. cill is fixed, and on top of this is laid the wedges, at least 3 in. deep, to facilitate the removal of the arch forms, which are framed of $1\frac{1}{2}$-in.

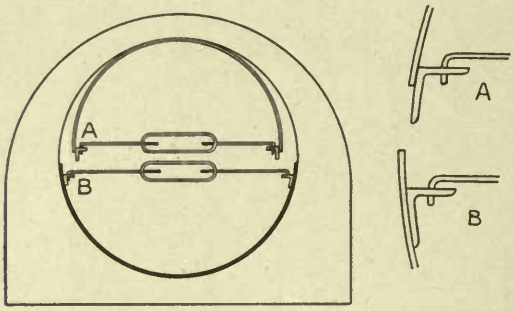


Fig. 345.—Collapsible Steel Centering for Sewer

stuff, 2 ft. 3 in. apart. The side forms can then be easily removed after the arch forms are struck.

Fig. 344 shows the form for a small box culvert 24 in. wide, the top splay giving it an arch effect cheaply. The outer form of the wall consists of 2 in. by 10 in. boards secured to 4 in. by 4 in. posts driven into the ground and braced by 1 in. by 3 in. cross pieces. The inner forms are of 1 in. boards propped up by 2 in. by 4 in. head pieces, and cills and posts, which are placed without nailing. The form is struck by pushing the 2 in. by 4 in. head pieces off the side posts.

Collapsible Steel Centering.—A great deal of sewer work is carried out by the Blaw collapsible steel centering, which consists of flexible steel plates bent cold to the proper radius and stiffened with channel ribs (as in Fig. 345). The sections are 5 ft. long, and held together at the ends by staples passing through slots on the male end of each section. The centres are held in position by two tension rods attached by a turnbuckle, these being also used for collapsing the centres. The rivets being countersunk present a fair face to the concrete. The following briefly describes the method of their use. A dish is prepared in the trench, on which to place the centre. The concrete is then placed in position and

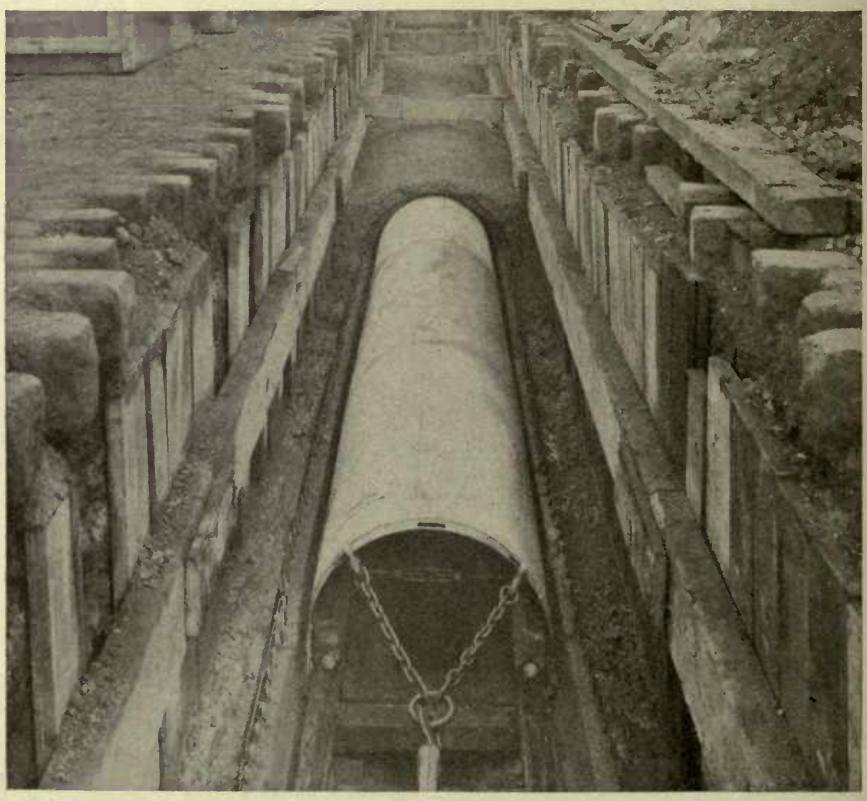
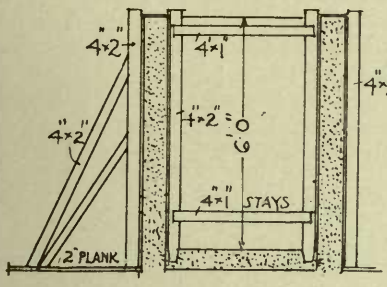
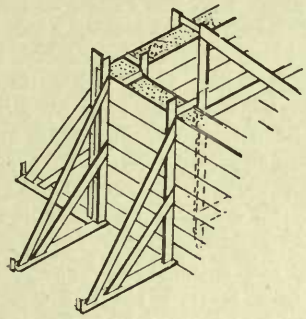


Fig. 346.—Conduit at Woolwich, showing Collapsible Steel Centering in Use

ammed; after it has set, the centres are ollapsed by means of the turnbuckles, which ase them from the concrete without any arring or hammering whatever. When





Figs. 347 and 348.—Form for Square Tank

he centres are collapsed they are dropped o rollers and pulled along ready for the ext section. When the centres are very arge they are mounted on small wheels run n light rails. The centering is adapted to

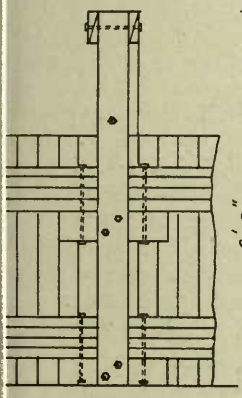


Fig. 349

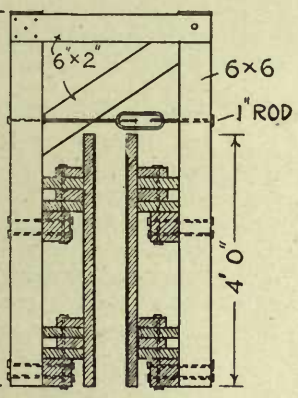


Fig. 350

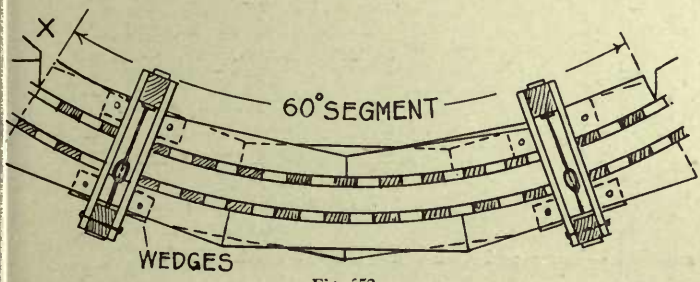


Fig. 352

Figs. 349 to 352.—Form for Circular Tank

ifferent shapes for all kinds of conduits nd for concrete pipe moulds.

Fig. 346 is a reproduction of a photo- raph clearly showing the Blaw co lapsible entering in use.

FORMS FOR TANKS

Form for Square Cistern.—Figs. 347 and 348 show forms for the erection of a small cistern 4 ft. 6 in. by 4 ft. 6 in. by 6 ft. high. 1-in. dressed boards are cut to the exact inside dimensions, namely 4 ft. 6 in., on two sides, and 4 ft. 4 in. on the other two sides, and they are held in position by

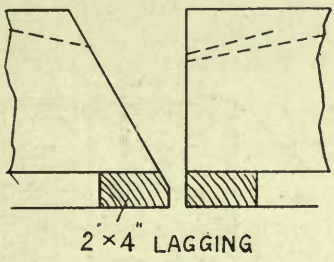


Fig. 351

2-in. by 4-in. posts strutted across with 1-in. horizontal boards at top and bottom. Eight 2-in. by 4-in. posts at the corners are set on the ground, and held in position by 2-in. by 4-in. inclined strutting nailed to planks on the ground secured by stakes. The inner and outer forms are kept the proper distance apart by 1-in. battens nailed to both posts. The posts may be much higher than the tank in preference to cutting the material. The outside boards can be allowed to project beyond the corners and thus save needless cutting.

Form for Circular Tank.—Figs. 349 to 352 show forms for a circular water tank 24 ft. in diameter. Six segments of 60

degrees each form the circumference, and they are 4 ft. high. There are two bands of spliced 2-in. by 8-in. planks at top and bottom shaped to the circle, and nailed to these at 8-in. centres are 2-in. by 4-in. laggings to which are secured No. 22 gauge galvanised iron. The yoke, formed of 6-in. by 6-in. verticals, 2-in. by 6-in. braces and adjusted by a turnbuckle, is the means by which the outer and inner forms are kept the proper distance apart, it also enabling the forms to be raised as the work proceeds.

Form for Gasholder Tank.—Fig. 353 represents a section through the walls of the gasholder tanks at Dubuque, erected for the Key City Gas Company. The bottom is about 5 ft. below the outside ground level, and the walls rise to a height of 21 ft., being 18 in. thick at the base, tapering to 12 in. at the top. The concrete was first laid over the entire bottom 16 in. thick, but increased under the walls to a thickness of 2 ft. 6 in.

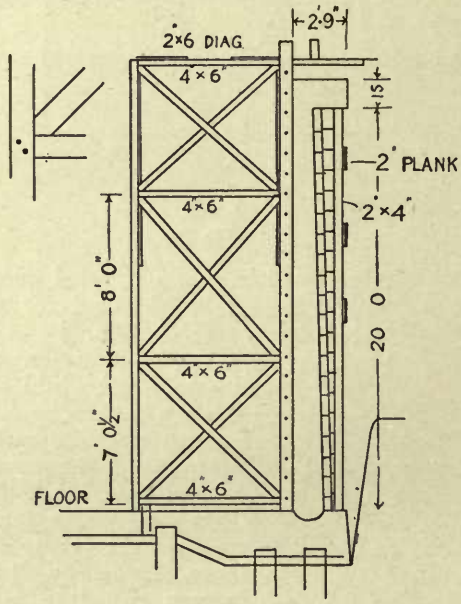


Fig. 353

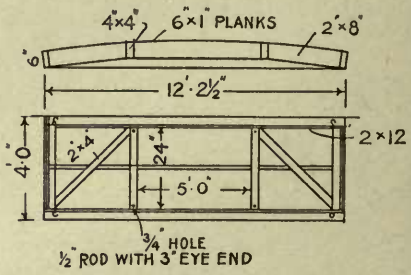


Fig. 354

Figs. 353 and 354.—Form and Centering for Gasholder Tank

As the foundation was uncertain, piles were driven 4 ft. 6 in. apart to support the floor. The last named was roughly dished at the bottom of the wall in order to give a good key both to the bottom of the wall and to support the scaffolding of the wall forms, which were fixed to temporary posts driven where shown to carry them. The concrete round the heads of the posts was also dished out in order that the posts could be cut down in the striking of the scaffolding, and the holes filled up. The whole 264 ft. circumference of the tank wall was filled up to the level of the top of the form in one operation. Forms were used to make a mortise and tenon joint between one day's work and the following day's. Between the outer side of the wall and the earth, up to the higher ground level, concrete was filled in, well rammed and tamped, and it was relied upon as a support to the base of the wall. Two 2-in. by 12-in. planks nailed on top of temporary posts were laid radiating to the tank wall. On top of this two layers of 1-in. by 6-in., spliced in short lengths and shaped to the radius of the wall, were fixed, braced in position with 2-in. by 6-in. uprights and 2-in. by 4-in. inclined braces. The scaffolding supporting the wall forms on the inside of tank consisted of 4-in. by 6-in. cills laid on the ends of the post, which projected 8 in. above the floor; from these were erected double 2-in. by 4-in. verticals, with 4-in. by 6-in. ledgers and 2-in. by 6-in. cross bracing. The outside alternate pilasters and piers were built up with two sides complete, and the third side added as the work proceeded. The shuttering to the outside wall was similar to that on the inside and was fastened in position between and under the edges of the pilaster forms and lag screwed to the 4-in. posts. The edges of the inside shutters were bolted to the sides of the inside 2-in. by 4-in. verticals of the scaffolding, and the little triangular spaces between the shutters were filled up with 2-in. vertical strips of steel on a backing strip of wood 1 in. square.

The forms were raised 3 ft. at one operation, by blocks attached overhead to a cap piece, and as the forms were 4 ft. high

there was an overlap of 1 ft. on the finished concrete. The forms were cleaned and oiled after every operation. Fig. 354 shows the inside shutter for the tank.

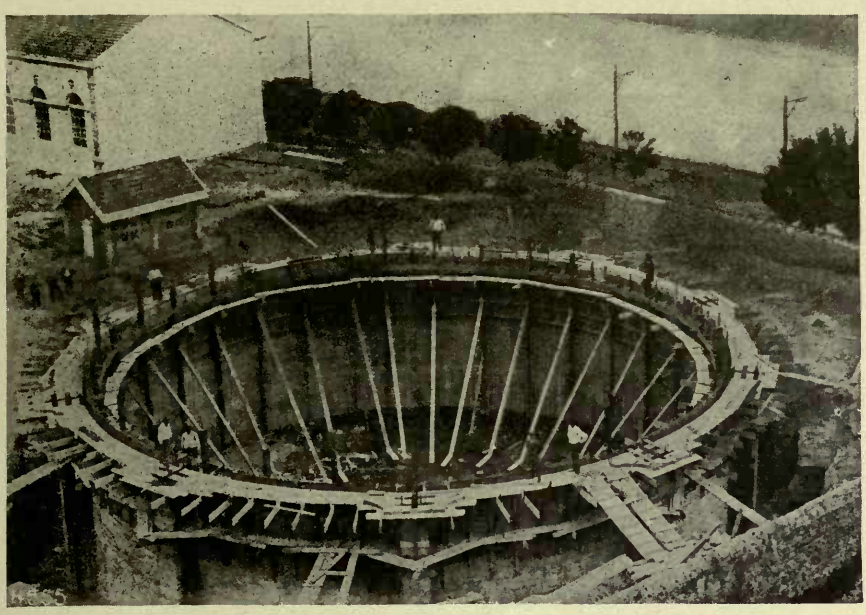


Fig. 355.—Form for Gasholder Tank at San Sebastian

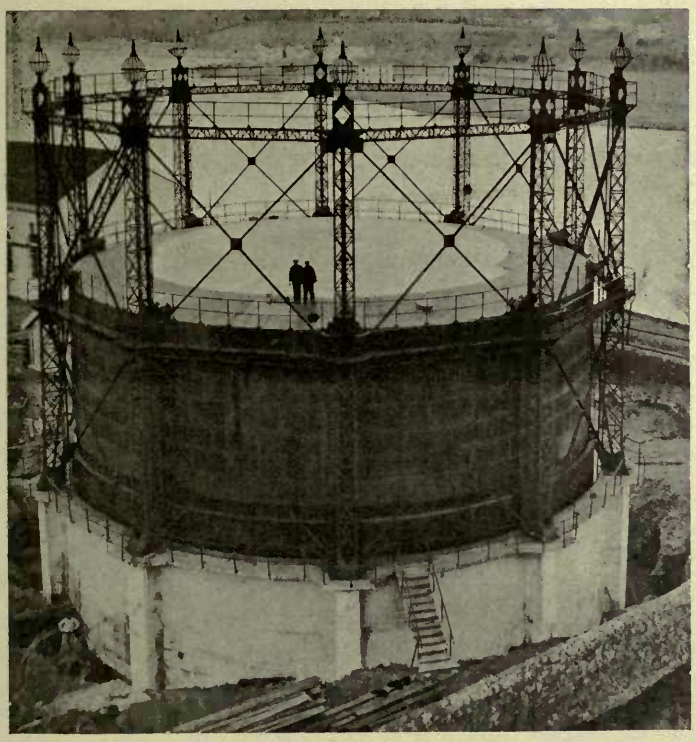


Fig. 356.—Gasholder Tank at San Sebastian

A photographic view of the forms, etc., for a gasholder tank at San Sebastian is presented by Fig. 355, the finished tank being shown by Fig. 356.

Form for Rectangular Reservoir.—In an example of a rectangular reservoir having walls strengthened by pilasters, the slab roof was carried on rib beams. In excavating for the walls, the footings were made slightly larger and a 2-in. plank on edge inserted. After placing concrete for the footings, horizontal shuttering of 2-in. board, dressed inside and propped up by 4-in by 4-in. uprights placed at 4-ft. centres, was erected. For the curved corners thin boards were bent to the required curve and secured to the uprights, the latter being 2 ft. apart. The posts on

the outside were propped up by raking struts and wires passing through the wall to the inside, there being braces connected to the adjoining columns and walls, and also struts extending to a proper bearing. As before, the two sides of the pilasters were completely formed, and the third side added as the work proceeded. The slab ceiling and beam forms were carried out as previously described for floors, with 2-in. boards dressed on one side, both to the beams and slab. The beam forms and slab centering were erected on 4-in. by 4-in. uprights resting on the finished floor beneath and cross braced at intervals.

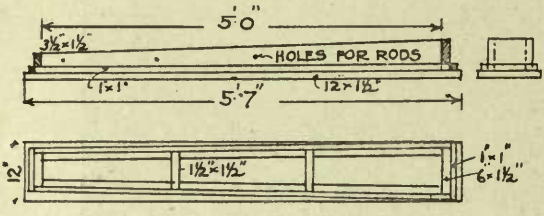


Fig. 357.—Elevation, Plan, and End View of Form for Tapered Square Posts

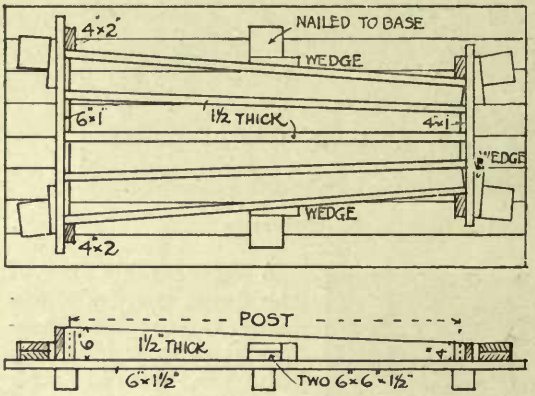


Fig. 359.—Plan and Elevation of Multiple Form for Tapered Square Posts

FORMS FOR FENCE POSTS

Reinforced concrete fence posts have been used with success, and although their first cost is more than wood, in the end they are cheaper, as they require no upkeep. Figs. 357 and 358 show the form for square posts. A plank is laid as a base dressed on the upper side, and on this are set two tapering pieces, 2 in. by 6 in. at one end tapering to 2 in. by 4 in. at the other. These tapering pieces are held in position without nails by the side fillets which are nailed to the base board. The tops of the sides are stayed as shown. Fig. 359 shows a multiple form for moulding four fence posts at a time, the posts being tapered four sides. The divisions separating the posts are slipped in between cleats at each end, and held in place by wedges against blocks nailed to the platform. The illustration is self-explanatory. Fig. 360 shows a mould for triangular posts. The frame is hinged so that posts having different angles can be made in it.

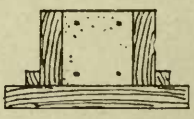


Fig. 358.—Cross Section (enlarged) through Form for Tapered Square Posts

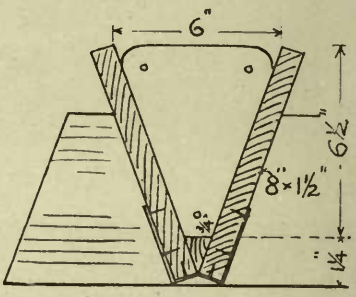


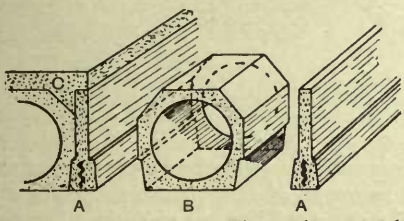
Fig. 360.—Section of Form for Triangular Posts

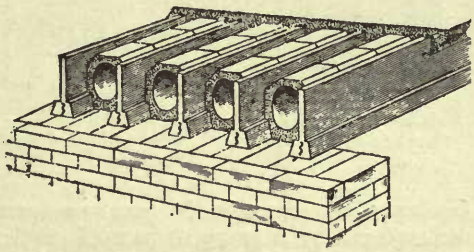
Systems Described

In this chapter will be found descriptions of a number of the best-known modern systems of reinforced concrete, based on information supplied by various firms in the industry. It will thus be understood that where special claims are made for this or that arrangement, it is simply the inventor's or owner's point of view that is presented. The explanations have been kept as concise as possible, since, as most readers already know, the firms responsible for the various systems issue lengthy and well-illustrated descriptions, which may generally be had for the asking, and it is therefore a simple matter for the reader to follow up any special claims or other points of interest here briefly stated which may invite his further study.

ARMOURED TUBULAR FLOORING

A typical floor on this system consists essentially of three parts (*see* Figs. 361 and 362), the concrete webs A, coke-breeze tubes B, and a concrete top layer C. The webs are reinforced with a ribbed corrugated bar as shown, and these, together with coke-breeze tubes, which are 9 in. long, are delivered complete on the site. First the webs are placed in the proper position, then the tubes are put in between, and the top layer of concrete is spread over the whole. The ceiling side of the floor may be left as laid, or may be finished by plastering.





Figs. 361 and 362.—Armoured Tubular Floor

CLINTON SYSTEM

Slabs.—This system depends on the use of an electrically cross-welded steel wire fabric, consisting of a series of parallel longitudinal wires spaced at certain distances apart, and held at intervals by transverse wires, the two being electrically welded together at the points of intersection. The wire fabric can be supplied as bright steel or galvanised; either of the strands may be of any thickness, from No. 4 to No. 12 gauge (Imperial Standard Wire Gauge), but No. 5 is the heaviest that can be used when both strands are of the same size. The longitudinal wires are spaced 3 in. apart, and the standard spacing of the transverse wires is 12 in., 16 in., or 18 in. The maximum number of longitudinal wires is 24. The standard maximum width is 72 in. The greatest width possible is 97 in., and the wire fabric is supplied in rolls or sheets, a roll of the heavy gauge containing 150 ft., and of the lighter gauges 200 ft.

COIGNET SYSTEM

Beams and Slabs.—The essentials of a Coignet beam are shown by Fig. 363, in which A indicates the principal bars subject to tension, B secondary bars working in compression, C stirrups to resist shear, D principal bars in floor slab, and E secondary bars in floor slab. The principle of the beam is the introduction into the compressed portion of the concrete of the secondary bars over which the stirrups or shear members are hooked. The stirrups or shear members, which connect together the concrete working in compression and the bars working in extension, increase the resistance to shearing and also the compressive resistance of the concrete. Where the stirrups pass round the bars, they are fastened by means of annealed wire so as not to be disturbed when the concrete is applied. The skeleton reinforcement is

prepared in advance, and stacked till ready for use. As the spacing of the stirrups is proportional to the shearing stresses, they are closer together near the supports where the maximum shearing stresses occur. Much importance is attached to the fact that both stirrups and the main bars are of round section, so that the line of contact between them is a mere line.

Fig. 363 illustrates an early type of beam, which has been largely superseded by that shown by Fig. 364.

In a later type of Coignet beam (Fig. 364) the lower bars shown in the previous figure are replaced by a group of bars of smaller diameter, their ends being bent upwards at an angle of 45 degrees and hooked over the top bar. The sectional area of the reinforcement is greatest in the middle portion of the beam where the bending moments are higher, and the bars are spaced closer together near the supports as in the previous example. The bars are bound together with annealed wire as before. The bars do not lie in contact one with another, there being a space of at least $\frac{1}{4}$ in. between them into which the concrete flows.

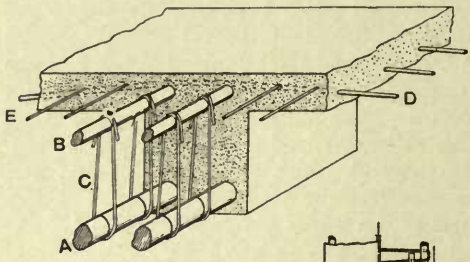
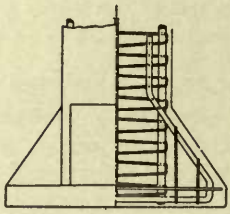


Fig. 363.—Coignet Beam and Slab



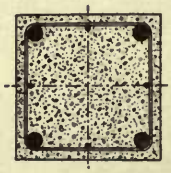


Fig. 366.—Section through Coignet Column

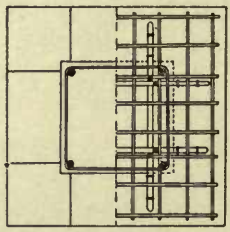


Fig. 367 and 368.—Base of Coignet Column

The reinforcing of the floor slab has already been shown in Fig. 363. The principal bars D are of such diameter and spacing as will best resist the tension, the secondary bars E merely distributing the efforts more easily on the principal ones. At alternate intersections of the bars, they are bound together with annealed wire. Coignet floor slabs usually vary in thickness between 3 in. and 6 in.; but should the thickness exceed this, a double reinforcement is advised, stirrups connecting the upper and lower bars at intervals.

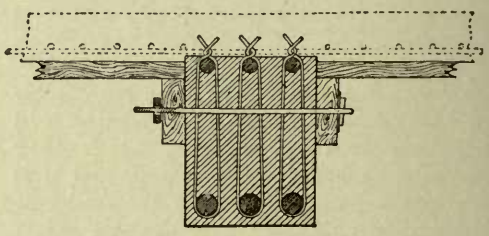


Fig. 365.—Beam Supporting Floor Slab Centering

Fig. 364.—Coignet Beam Reinforcement Consisting of Group of Small Bars

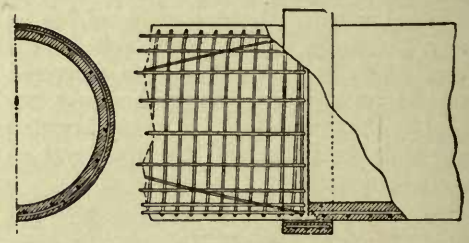


Fig. 369 and 370.—Coignet Pipe or Conduit

Coignet beams may be prepared in advance and transported and placed in position like steel girders. They are made in a horizontal wooden mould, the principal bars being left protruding for about a foot at each end so that they may be fixed on to their supports by means of cement grout. Fig. 365 shows how the ready-made beams may be used to support the centering for the floor slabs. The upper ends of the stirrups project so as to form a mechanical bond. This method is seldom used.

Columns.—A simple type is shown by Fig. 366. In this, the vertical bars are

bound by horizontal or spiral hoops or ties to keep them in their proper position and also to resist a bursting tendency. The hoops are fastened to the verticals by means of annealed wire. The footing of the pillar where it is necessary to spread the weight takes the form shown in Figs. 367 and 368.

Walls.—As these are to resist lateral pressure only, and not intended to support any vertical load, this being carried by beams, a meshwork of vertical and horizontal bars is placed in the centre of the wall to facilitate concreting, the intersections being bound together with wire.

Pipes.—The reinforcement takes the form shown in Figs. 369 and 370, and the pipe or sewer is constructed in lengths of about 4 ft. or 5 ft., and connected to the next section by means of joint and spigot as shown; alternatively, the pipes may be concreted in continuous length.

Piles.—Practically, these are pillars made in a horizontal mould instead of a vertical one. Generally, as in Fig. 170 (p. 133), they are circular in section, with two flat longitudinal surfaces which assist the proper guiding of the pile during driving.

CONSIDÈRE SYSTEM

Beams and Slabs.—This system is well-nigh devoid of fanciful methods of shaping and arranging the reinforcement, and is claimed to rest entirely on a scientific basis. In the case of minor beams, the tensional reinforcement takes the form of round bars, some of which are horizontal throughout, and others are bent up at the ends. Where a compression member is required, a spiral coil of round steel is inserted near the top surface. Occasionally beams are additionally reinforced against shear by means of thin steel rods, which are lapped round the tension and compression bars. In addition to the above, Considère sometimes inserts an extra reinforcement (a spiral coil of round steel laid nearly horizontal) in that part of the concrete which is in compression near the supports of a continuous beam. The slab reinforcements between the beams consist of round bars bent up over the supports at each end, there being, in addition, round bars which cross the other reinforcements at right angles. In large girders and trusses, as employed in bridge construction, the tension members are reinforced in the manner already described, whilst the compression members are reinforced with longitudinal bars and spiral coils.

Columns.—Columns are generally circular, octagonal, or square in cross section, and the vertical reinforcements are straight round bars from 4 in. to 6 in. apart, whilst the transverse reinforcements consist of round rods in the form of coils wound spirally round the vertical reinforcements, the ends of the coils being bent inwards.

Piles.—Considère has devoted much attention to the construction of reinforced concrete piles on scientific principles. His general form of pile is octagonal, with longitudinal reinforcing rods, generally numbering about eight, inside a continuous spiral winding of round steel rods, the pitch of the spiral being about 2 in. at the middle and diminishing to 1½ in. at both head and foot. The head is of cylindrical shape and for about 4 in. is bound with steel coils closely pitched, this reinforcing the head to such an extent that it will not fracture under the driving shock, even though a cap or dolly is not used. Illustrations showing the application of the Considère system in practice will be found on pp. 358 to 360.

CORR BAR

This is an American system of supplying beam and column reinforcement already made up and suitable for immediately placing in the forms. Each made-up reinforcement is called a "unit," and a typical construction is shown by Fig. 371, this type having been designed to give, by means of continuous stirrup, an efficient web reinforcement. The unit for a round column consists of two or more verticals with a continuous hooping of cold-drawn wire. The American company responsible for this system has introduced a variety of special reinforcements, including deformed bars (*see* Fig. 372) and a kind of expanded steel known as "Corr Mesh."

Fig. 371.—Corr Bar Beam "Unit"

Fig. 372.—Types of Corr Bars

DENTILE SYSTEM

Floors.—By means of this system, floors are constructed having the advantages of terra-cotta tile floors and those of reinforced

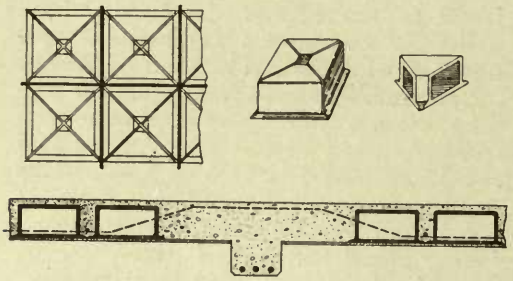


Fig. 373.—Dentile Floor with Mitre Tiles

concrete. Briefly, upon the forms are placed hollow tile blocks closed on all six sides and having projecting flanges, which space them at proper intervals. In the channels so left the reinforcing bars are placed, the concrete being then applied and a floor surface being obtained by spreading a layer of concrete

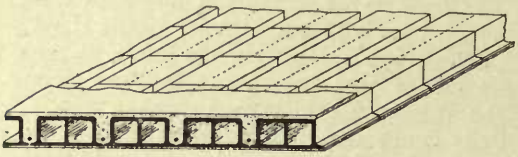


Fig. 374.—Dentile Floor with Bridge Tiles

right over the tiles. Essentially, then, the floor consists of hollow tiles with ribs of reinforced concrete between them united at the top by a thin slab of plain concrete. On the ceiling side there is a continuous surface of tile. Clear spans up to 32 ft. have been built and tested, while for light loads,

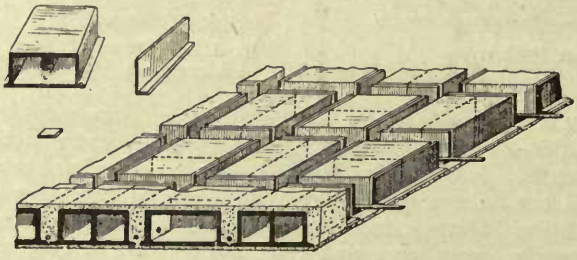


Fig. 375.—Dentile Floor with "L" Tiles

and where surrounding conditions are favourable, the spans may be as much as 40 ft. Three shapes of tile are used: mitre tiles (Fig. 373), consisting of four wedge-shaped portions fitting together to form a closed hollow block. The reinforcement can be placed in both directions. In the case of

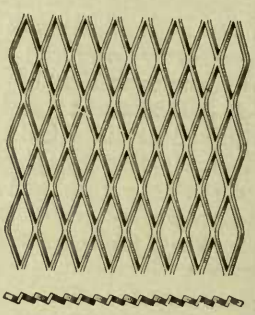


Fig. 376.—Diamond Mesh Expanded Metal

the bridge tile, shown by Fig. 374, the reinforcement is in one direction only. With L tiles, (Fig. 375), the reinforcement can go both ways. Small flat filling pieces are used to put in the square corners between the tiles. In all cases the under-sides of the tiles are roughened so as to take the ceiling plaster.

EXPANDED METAL SYSTEM

Expanded metal consists of rolled steel of various thicknesses, cut and expanded by machinery into meshes of various shapes, the material being obtainable in a number of different strengths or weights. Diamond mesh is shown by Fig. 376. There are thirty varieties of this material, the variations being in the size of mesh, thickness of strands, the

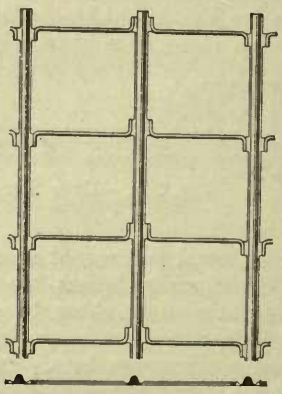


Fig. 377.—Rib Mesh Expanded Metal

weight per yard super, etc. The rib mesh is shown by Fig. 377, this being made in five varieties; however, for concrete work the diamond mesh material is chiefly used.

What is known as the expanded steel bar is a bar from which a series of meshes has been expanded (*see* section, Fig. 378). It is claimed to be a complete unit, comprising tension and shear reinforcements, and as its members are rigidly connected, they cannot get out of place during the concreting. When expanded metal is used in connection with these bars, the ends of the sheets may be wired to or bent over and lapped on to the web of the bars, thus securing continuity of reinforcement.

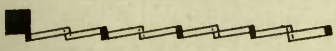


Fig. 378.—Section of Expanded Steel Bar

Floors.—When these bear on the walls, it is necessary to leave an offsett, corbel, chase, or over-sail. The method of constructing the floor with expanded metal is described on page 152, and Fig. 379 presents sectional drawings of a few typical styles of construction.

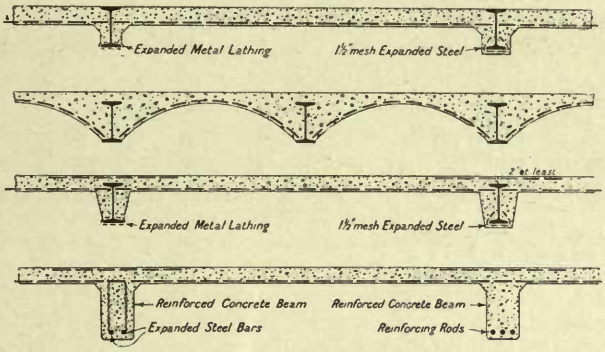


Fig. 379.—Four Types of "Expanded Metal" Floors

Other Applications.—Expanded metal has been found applicable to a wide range of work which it is quite unnecessary to describe in detail.

HENNEBIQUE SYSTEM

Beams.—Scientific designing rather than the use of steel of special shape or high-carbon content is the essential of this system. Thin bars or rods and stirrups of steel are inserted in just those places where the stress diagram of the structure shows that the resistance of the concrete requires to be supplemented. The experts responsible for the system believe that if it were practicable to apply the teachings of theory to their full extent, the steel reinforcement employed would be in the form of fine wires receiving and transmitting stress throughout the concrete. They therefore endeavour to reconcile theory and practice by using moderately small bars as the main reinforcement, and small rods, wire, or steel strips as the connecting or auxiliary reinforcement, and all these are in forms procurable in the open market. They believe that a beam reinforced simply by straight tension bars near the lower face would not be satisfactory in practice, and therefore they supplement those bars—in a beam of the most simple type—by a number of stirrups clipped firmly around the tension bars as shown in Fig. 380 and bent over at the top so as to be anchored into the concrete, and they also insert other bars straight for the greater part of their length and bent upwards at each end, part of such a bar being shown by Fig. 381. Diagramatically, a simple beam of this type is shown in Fig. 382, which includes a longitudinal section and three cross sections. The bars near the lower surface adequately reinforce the middle portion of the beam, and the inclined bars and the vertical stirrups supply the additional reinforcement necessary at the ends of the beam where tension occurs at diagonal planes. The company emphasises their belief that the bars and stirrups reinforce the concrete against diagonal tension rather than against vertical shear, and say that what, in beam tests, is commonly described as a "shearing" failure, is almost invariably a diagonal tension failure. The bent-up ends of the bars lie across the lines of rupture near the supports of the beam, and afford in themselves very secure anchorage. The spacing of the vertical stirrups is such as to provide for variations of stress from point to point. Being vertical, they facilitate the operation of ramming the concrete without risk of displacement, and being made with a spring clip at the lower end, they are automatically held in position on the main bars without either wedges or ties. The stirrups form an effective web connection between the tension and compression portions of the beam. The company does not believe that any stress or vibration developed under working conditions can possibly overcome the natural adhesion between the concrete and the plain round bars, but the ends

of the bars are flattened and opened out to form secure anchorage, and at least half the bars are bent up towards the supports, further to ensure perfect security. In beams that are continuous over intermittent supports, the ends of the bent-up bars are carried horizontally across the supports and terminate near the points of contraflexure (*see* Figs. 383 and 384). The inclined bars in each span, and the horizontal ends of other inclined bars projecting from the adjoining spans, then resist the tensile stress occurring in the upper part of continuous beams between the supports and the points of contraflexure.

Beams constructed as above described are the most economical types, but it is sometimes advantageous to employ compression bars near the upper surface as shown in Fig. 385, which type of beam has advantages when a very heavy load has to be carried by a beam of comparatively small dimensions. The double stirrups will be noticed.

Floors.—Hennebique floors are simple combinations of beams and slabs. The main beams receive the load which is transmitted from the secondary beams, the latter receiving the load transmitted from the continuous slab connecting all the beams and receiving the super load placed upon the floor. In a typical instance the main beams have parallel reinforcing bars, and the secondary beams have single reinforcement. In the usual type of Hennebique floor half the slab panel on each side of each secondary and main beam constitutes a compression flange, and, with the beam, forms a beam of T-section. The longitudinal bars and stirrups of the main beams pass into the superimposed slab, and where there is compression reinforcement, this is entirely within the slab. The Hennebique hollow-tube flooring is mentioned on p. 332.

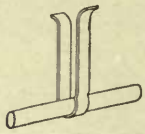


Fig. 380. — Hennebique Stirrup round Tension Bar

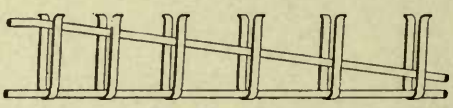


Fig. 381.—Hennebique Tension Bars and Stirrups

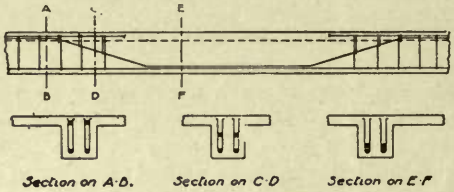


Fig. 382.—Simple Hennebique Beam

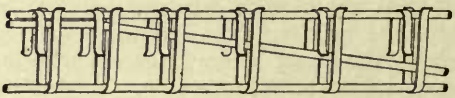


Fig. 385.—Beam Reinforcement with Compression Bar and Double Stirrups

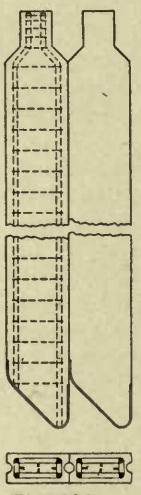


Fig. 389.—Sheet Piles

Fig. 390.—Hollow Diaphragm Pile

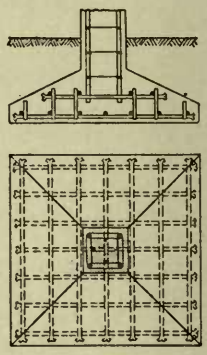
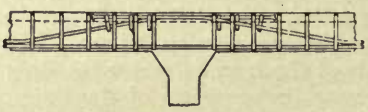
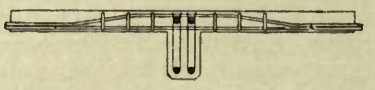


Fig. 387 and 388.—Base of Hennebique Column





Figs. 383 and 384.—Hennebique Beams Continuous over Intermittent Supports

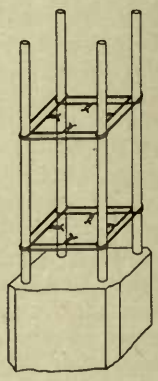


Fig. 386.—Hennebique Column

Columns.—A typical Hennebique column is shown by Fig. 386, the vertical reinforcement consisting of plain round bars, and the auxiliary reinforcement being formed of links of $\frac{3}{16}$-in. steel wire applied in sets of four, one link passing round the two verticals on one side. The links brace the longitudinal bars and support the concrete laterally. This method was introduced many years ago, and has given satisfaction in thousands of cases.

Figs. 387 and 388 show a column base, the horizontal steel reinforcement distributing the force transmitted by the vertical bars. The lower portion of the concrete is reinforced by a double system of bars laid at right angles with one another so as to provide for the tensile stresses caused by the bending moment developed by the central load and the vertical reaction of the ground. Resistance to compression is provided entirely by the upper portion of the concrete, and diagonal tension is taken by the vertical stirrups applied as in beams. A column base of this description virtually represents cantilever construction.

Piles.—These can be built of any approved shape, that shown on p. 133 being typical. The longitudinal bars are bent inwards at the toe, but do not bear upon the steel driving point, this being anchored into the concrete by four straps bent over at the upper end, as shown on p. 133. The transverse ties, resembling those in the column already described, are spaced more closely near the top and bottom of the pile than in the middle.

Sheet piles for retaining walls and cofferdams have a head of reduced section (*see* Fig. 389), and on two sides of the pile there is a semicircular groove, the hole formed by the grooves in contiguous piles being filled with cement grout to prevent the percolation of water. Thus a row of sheet piles forms a watertight wall. The hollow diaphragm pile (*see* Fig. 390) is constructed on the principle that the resistance to driving is largely, if not entirely, due to friction between the external surface of the pile and the earth, and that a given volume of reinforced concrete employed in the form of a hollow cylinder is more effective than if applied as a solid cylinder. The reinforcement is practically the same as that of the solid pile, but, in addition, diaphragms hold in place a consecutive series of tubes each about 4 ft. long, the object of the tubes being to form the hollow core of the finished pile.

INDENTED BAR SYSTEM

This system depends entirely on the use of a deformed bar, either square or round in section (*see* Figs. 391 and 392). The square bar varies from $\frac{1}{4}$ in. to 2 in. square, and from ·24 lb. to 13·6 lb. per foot run in weight, whereas the round bar varies from $\frac{3}{8}$ in. to $1\frac{1}{2}$ in. in diameter, the weight per foot run at these sizes being ·38 lb. and 6·06 lb. respectively. Strictly speaking, the bar itself is the "system," the company responsible for it having no special method of arranging the reinforcement, but preparing their designs in accordance with approved practice, and supplying the indented bars to the building contractor to be used in accordance with the designs prepared by the Indented Bar Company or by independent engineers. They contend that their bar obtains a more reliable grip on the concrete than does any smooth bar. The adhesion between smooth bars and concrete depends, they state, upon the cement particles entering into the microscopical irregularities in the surface of the steel. When a steel bar is subject to a tension of 12,000 lb., or say 5 tons, per square inch, it extends by ·004 in. per inch of length, this causing corresponding reduction in the area of cross section, which, although extremely small, is sufficient to affect the adhesion. They instance a number of tests, the results of which go to show that indented square bars sustain 1·9 times the stress required to pull out smooth squares, and indented round bars 3·7 times the stress required to pull out smooth round bars. Further, it is stated that owing to the fact that at a stress of 3 tons per square inch the extension in steel is greater than that to which concrete can be subjected without cracking, it appears to be undesirable when smooth bars are used to allow the tension in the steel to exceed that amount. With

Fig. 391.—Square Section Indented Bar

Fig. 392.—Round Section Indented Bar

indented bars, however, the continuous and positive bond provided makes it permissible for the stress in the steel to be limited only by the strength of the material used, this enabling a steel of high tensile strength to be employed. The draft regulations of the London County Council propose that there should be a mechanical bond between the steel and concrete in cases where steel used has a greater strength than 72,000 lb. per square inch.

When the company's ordinary specification is adopted, the bars are of rolled medium steel with an elastic limit of 50,000 lb. per square inch, with an extension of not less than 15 per cent. in a length of 8 in.; the breaking strength is 90,000 lb., roughly 40 tons per square inch. By the use of such strong material there is claimed to be a considerable saving in the weight of steel required for a given strength, the factor of safety can be increased without extra cost, and the omission of bending, splitting, cranking, and other devices commonly used for anchoring smooth bars is a source of economy.

JOHNSON'S LATTICE SYSTEM

This depends on the use of a steel wire mesh or lattice (*see* Fig. 393), manufactured in sheets or panels, and in rolls 6 ft. wide to about 120 ft., or the rolls can be made longer than 120 ft. and any width other than 6 ft., and the spacing can be varied to suit individual requirements. It is unnecessary

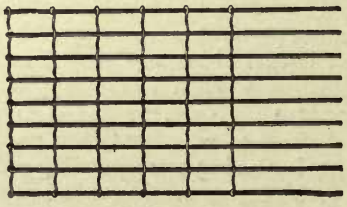
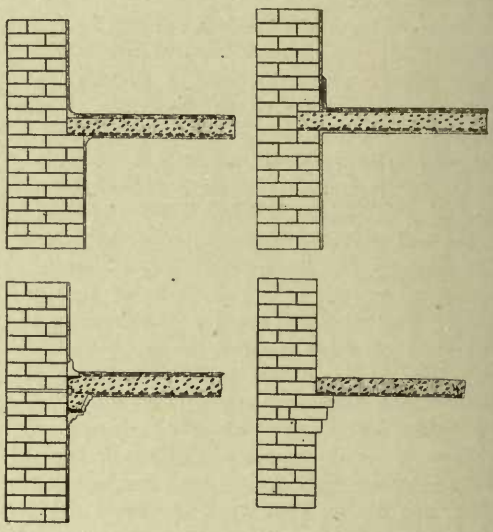


Fig. 393.—Johnson's Steel Wire Lattice

to give full details of the various methods in which this reinforcement is applied. It is sufficient to say that it is applicable to very many purposes, from foundation rafts, to floor slabs, walls, etc. Figs. 394 to 397 show four different methods of supporting mesh-reinforced floors.

KAHN SYSTEM

Beams.—The experts responsible for this system emphasise the fact that a properly constructed reinforced concrete beam is in reality a trussed beam, and they state that stirrups can only transfer stress to the main tensional member when the two are definitely and rigidly connected together, the horizontal reinforcement taking not only the stress caused from adhesion of the concrete to it, but also the summation of the horizontal components of the strain in each of the diagonals. The principles of truss action occur (or grow) out of this. The



Figs. 394 to 397.—Four Methods of Supporting Mesh-reinforced Floors

company does not believe that the concrete surrounding the bars will prevent the loose stirrups from slipping, and they therefore attach the shear members rigidly to the horizontal reinforcement. In the Kahn system a bar with two projecting wings or stirrups is used, the stirrups being bent up at about 45 degrees, so as to cross the planes of rupture at nearly right angles (*see* Fig. 398). At the centre span, where the horizontal tension is greatest, the web is left intact on the bar, and there serves as an additional reinforcement (*see* Fig. 399). The stirrups are part of the original metal of the bars, being merely sheared and bent, whereas in the original Kahn bar, holes were punched in it and the shear members fastened in these holes. The Kahn trussed bar, it will be understood, is sent to the site complete, ready to be incorporated in the concrete as a single unit.

The bar undoubtedly possesses great advantages in lintel construction. The principle is stated as follows: A flat arch acts

exactly upon the same principle as a segmental arch, and therefore if a section of the wall above an opening can be converted into such a flat arch, and a tie member suitably placed so as to receive its thrust, then that section of the wall has become in reality an arch. This, in short, is the purpose of the Kahn lintel, wherein the Kahn trussed bar is used as a tie member, the diagonals of the bar taking up the thrusts of the arch. The lintels can be built for spans as great as 30 ft. or 40 ft. Figs. 400 and 401 show the application of the Kahn bar in lintel construction.

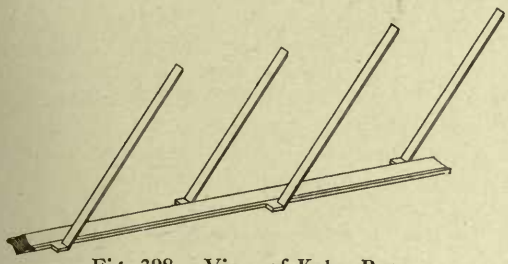


Fig. 398.—View of Kahn Bar

Fig. 399.—Section and Elevation of Kahn Trussed Bar

Floors.—The floor slabs, reinforced with top and bottom horizontal bars, the lower of which is a bar of the type already described, form, with a trussed beam, a beam of T-section.

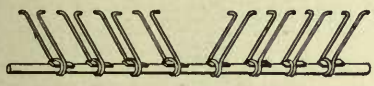


Fig. 402.—Keedon Beam Reinforcement

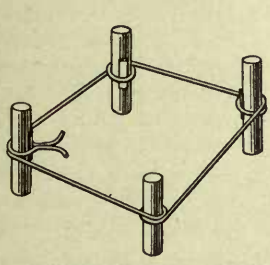
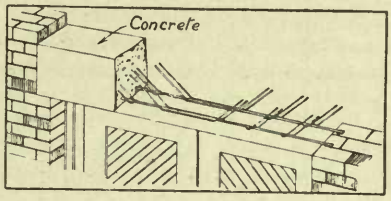
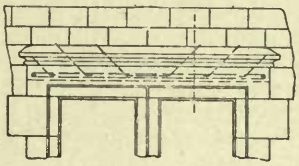


Fig. 403.—Keedon Column Reinforcement

Columns.—The Kahn bar is also applicable to column construction. The prongs projecting from the bars (the bars are here used as vertical members) reach diagonally across the column and tie in the main bars at intervals of 6 in. to 12 in. Some Kahn pillars have helical hooping.





Figs. 400 and 401.—Lintel Reinforced with Kahn Bar

"Hy-rib" is a sheet-mesh reinforcement and lathing introduced by the Kahn Company.

KEEDON SYSTEM

The peculiarity of this system is the use of stirrups, hoopings, etc., which are looped to pass over the main bars, and are rigidly held to those bars by means of wedges or keys. Beam reinforcement on this principle is shown by Fig. 402, and column reinforcement by Fig. 403. The rigid shear members or stirrups resist the diagonal tension. Thus, the stirrups and hoopings form in effect a series of rigid projections, but the bars, of course, retain their original cross section, and are not deformed in any way. All the stirrups, hoopings, and keys are inter-

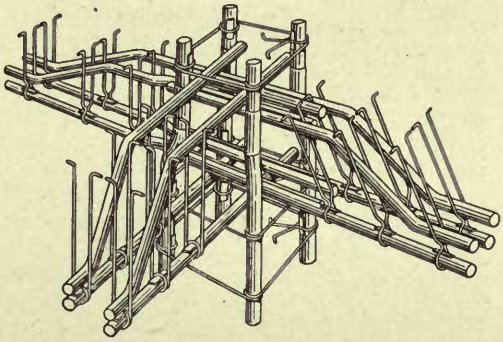


Fig. 404.—Column and Beam Reinforcements Keedon System

changeable, and the work of assembling them is quickly done. The main reinforcements are ordinary merchantable steel bars. Fig. 401 shows the Keedon system applied to a column and beams.

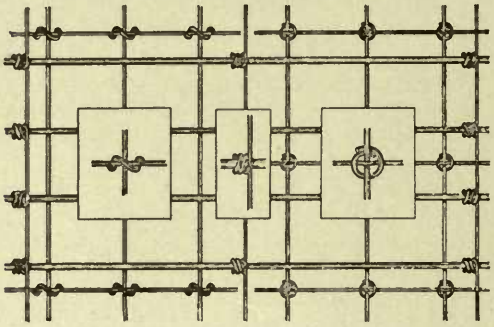


Fig. 405.—Lock-woven Mesh

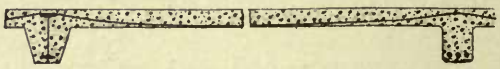


Fig. 406.—Lock-woven Mesh Floor

LOCK-WOVEN MESH SYSTEM

This depends on the use of a special reinforcement, consisting of steel wires woven together at right angles and secured at the intersections by means of machine-made

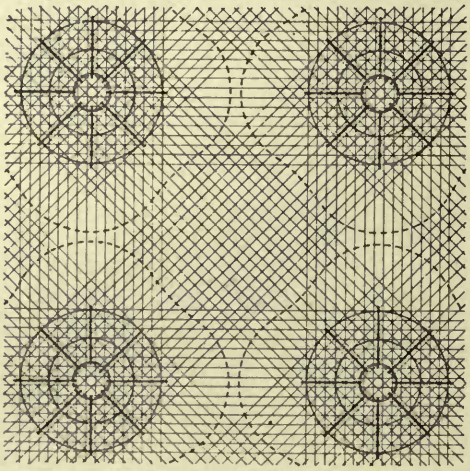


Fig. 408.—Floor Slab Supported by Four Columns, Mushroom System

knots, which are of three kinds, as shown in the composite diagram, Fig. 405. The lengthwise wires, known as carrying wires, are heavier than the transverse or distributing wires, since they take the tensional stress, whereas the other wires do little more than keep the fabric rigid in the course of laying and distributing any accidental loads and temperature stresses. The fabric is sent out in rolls or in the form of sheets cut to size and shape. The firm responsible

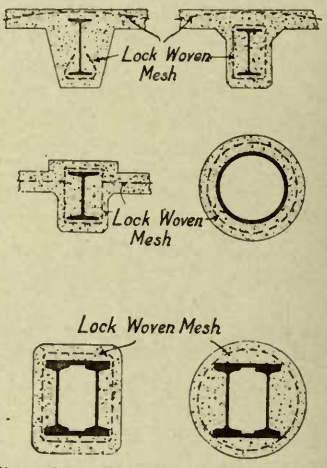


Fig. 407.—Fireproof Construction with Lock-woven Mesh

for the system emphasises the advantage of using the close-fibre steel, which is drawn out under considerable pressure, rather than a material in which the fibres have been severed by slotting. The method of placing

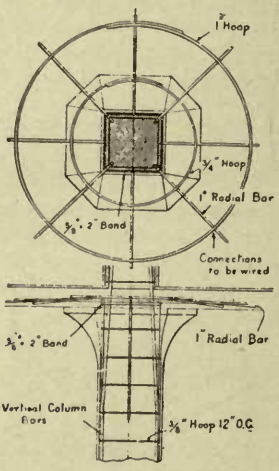


Fig. 409.—Head of Column, Mushroom System

in position reinforcements of this type is explained on p. 152 in the chapter "The Erection of a Reinforced Concrete Building." Fig 406 shows a type of floor, with alternative methods of constructing the beams. The reinforcement is carried over the steel joists

and sags in the centre of the floor, so that it comes close to the under-surface of the latter. It is obvious that Lock-woven Mesh is applicable to a great variety of reinforced concrete constructions. Fig. 407 shows methods of fireproofing floor girders and columns with reinforced concrete casings.

MUSHROOM SYSTEM

This is an American system, in which the bar reinforcement to the columns are splayed out radially top and bottom so as to be thoroughly bonded into the floors. As shown in Fig. 408, there are four belts of rods crossing the slab from column to column, and splayed out over the supplementary cantilever reinforcement at the top of the column (this description is based on one by the inventor, C. A. P. Turner, in his "Concrete Steel Construction," Part I.). It will be noted from the diagram that in certain areas the rods are only one layer or belt in thickness, and a practical test of the construction up to the yield point of the steel after the concrete has thoroughly set shows the development of cracks, due to the stretch of the steel, approximately along the dotted lines shown in the diagram, these lines therefore showing the planes of greatest weakness, and between them, in the centre of the slab, there being an approximately circular flat plate. The inventor demonstrates that these lines are nearly or approximately the points of maximum moment in the slab.

The Mushroom system is so called from the peculiar formation of the rods around the column head, and is claimed to simplify centering. It concentrates the maximum amount of reinforcement around and over the support where the shear is the greatest, and eliminates beams and ribs, giving a flat ceiling which allows of freer illumination from the windows and more convenient placing of shafting in the case of factory equipment. The system has been built and tested for nearly all spans of from 14 ft. to 30 ft., and larger spans can readily be made. Fig. 409 shows the head of a column, and is an example taken from actual practice. The columns are spaced 20 ft. 6 in. centre to centre each way, and the floors are designed to carry 150 lb. per square foot live loads. Each column is reinforced with eight 1-in. round steel bars, which project 4 ft. at the top, where, however, they are bent at right angles over a $\frac{5}{8}$-in. by 2-in. band placed just above the bottom of the floor slab. The rods flare radially into the slab from this band, and extend outwards to a distance of 3 ft. 9 in., while two circles of steel rods, 8 ft. 4 in. and 4 ft. 6 in. in diameter respectively, rest on the radial bars and are wired to them. The reinforcing bars in the floors extend well over each column head, and run parallel and diagonal to the lines of the columns.

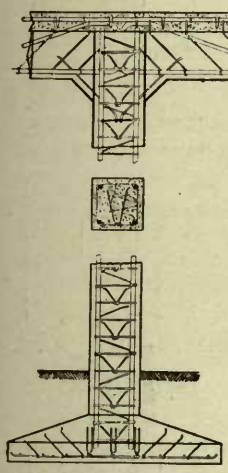


Fig. 411.—Paragon Column, Beam, and Floor Slab

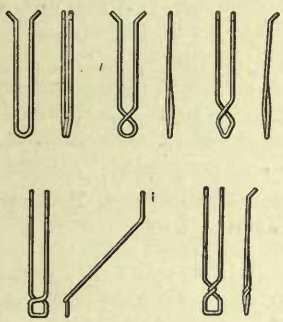


Fig. 410.—Paragon Stirrups

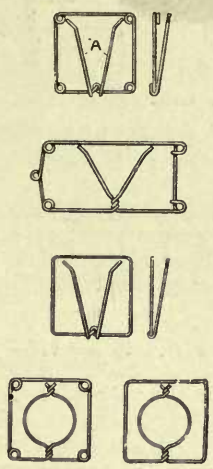


Fig. 412.—Paragon Column Hoopings

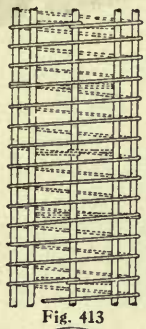


Fig. 413

Fig. 414

Figs. 413 and 414.—Paragon Helical Column Wrappings

PARAGON SYSTEM

Special forms of stirrups, hoopings, and wrappings are the peculiarities of this

system. Dealing first with the stirrup for use in beams, heavy floors, and similar structures subject to flexure or bending loads, the stirrups are relied upon to resist the web or shearing stresses; they lock tightly to the main bars, and are claimed to give a perfect mechanical bond distributing the stresses along the full length of the beam or slab to ensure that the concrete and steel act together in taking the load. They can be made to lie at any angle to make the most suitable form of truss and of any required length. They are so shaped as to give a minimum metal-to-metal contact, and they are available in a variety of shapes, in which Fig. 410 shows the principal five. Fig. 411 is a comprehensive diagram which includes a beam in which the stirrups are employed.

Hoopings are used in columns, piles, etc., and are claimed to reinforce the core in the planes between the hoops against the bulging action, for which purpose they have inturned arms, or their arms are twisted together to form spiral binds which lie through the centre of the core. Each hoop is placed so that its arms lie in a different direction from those of the adjoining hoops. Five styles of hoopings are shown in Fig. 412, and of these the first and last may be referred to. The first is for solid work, and has four bars which pass through the loops shown. The arms A extend across the area enclosed by the hoop and diverge, their ends being turned and bent away from each other. The last has the arms curved to form a central ring for use where a central column or pipe is to be enclosed.

The helical wrappings (Figs. 413 and 414) are claimed to overcome objections to which a continuous wrapping is liable—namely, that the concreting cannot be seen until it reaches the top of the column form; that concrete has to be dropped in from the top, giving the larger particles a chance of leaving the finer ones, and so forming a porous space; and, thirdly, the wire wrapping may be injured in the course of tamping, and so cause the whole length to be materially weakened. The Paragon helical wrapping has been sectionised, the ends of the sections being made to reinforce the core in the form of bonds similar to the hoopings already described. A wrapping is threaded by means of its loop over the end of a bar, fixed in position, and the concrete placed around it,

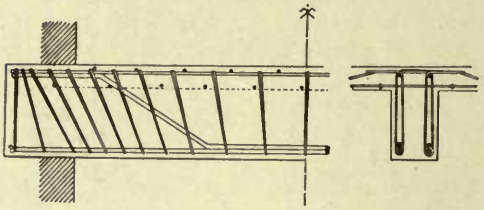
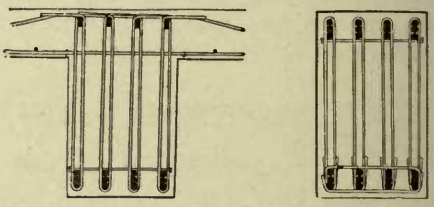
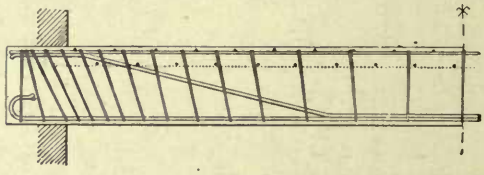
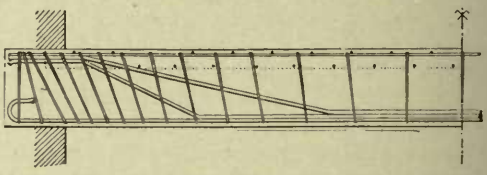


Fig. 415.—Piketty Beam

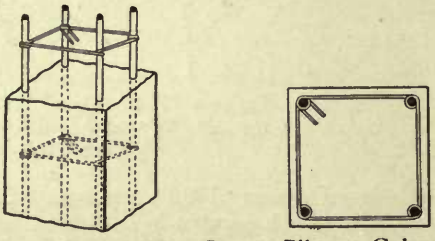


Figs. 418 and 419.—Cross Sections of Piketty Beam with Four Rows of Bars

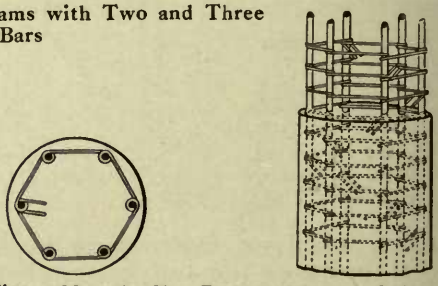




Figs. 416 and 417.—Piketty Beams with Two and Three Tension Bars



Figs. 420 and 421.—Square Piketty Column



Figs. 422 and 423.—Round Piketty Column

the tamping being done from the open side of the form and consequently providing no excuse for injuring the wrapping. The figure illustrates the reinforcement for a circular column, but square columns also can be made by this method.

PIKETTY SYSTEM

Beams and Slabs.—Paul P. Piketty, the inventor of this system, considers that double reinforcement is absolutely indispensable, for reasons which are pursued at length in the "Handbook" describing the system, among them being fastening of stirrups—increase of compressive resistance, and resistance to secondary tensile stresses being the result of permanent deformations in concrete. Fig. 415 shows two views of the reinforcement of an ordinary beam. There are two series of bars, namely, straight bars, and bars bent up at a third of their length. To resist the shearing stresses, there are stirrups connecting the lower with the upper reinforcing bar and inclined as shown; their ends are bent over as shown in the cross section, and all bars are fish-tailed to resist any tendency to longitudinal sliding. The beam shown by Fig. 416 is on the same lines, but the lower bar is hooked and fish-tailed. There are three series of bars in the beam shown by Fig. 417, whilst Fig. 418 is the cross section of a beam with four rows of bars. Fig. 419 is the cross section of a beam with symmetrical reinforcement. The transverse bars are in tension, whilst the horizontal links hold the four rows of bars together and increase the compressive resistance of the beam. Slabs on this system have both lower and upper reinforcement, the bars of one being generally at right angles to the bars of the other.

Columns.—Figs. 420 to 423 show pillars or columns reinforced on the Piketty system. The square pillar has four vertical reinforcements, and the special shape links will be noted. The round column has six reinforcements, the links being on the same principle as in the square column.

OTHER SYSTEMS

Bonna Pipes and Conduits.—These are reinforced with ribbed bars formed into longitudinals and spiral coils, the two being notched together and tied at the joints (*see also* p. 369).

Chain Concrete Floors.—By means of special clips, round steel bars laid parallel in the floor slabs are connected together to form a continuous sheet.

Columbian Floors.—The reinforcement comprises ribbed steel bars in conjunction with rolled steel joists, or heavier section ribbed steel bars alone.

Dawnay Floors.—The reinforcement consists of rolled steel joists at 16-in. intervals, or square bars at 12-in. intervals, laid between other joists.

Ellis Pipes.—Round steel rods reinforce sections as short as 2 ft. or 3 ft., there being an ogee joint to each section.

Hodkin-Jones Floors.—These are reinforced with corrugated bars placed on edge and resting on steel girders through the medium of a bent and slotted plate.

Homan Floors.—These are of many types. In one, the webs of rolled steel joists are pierced to allow of the passage of round bars which project into adjoining slabs; and in another, the reinforcement takes the form of a T-bar with corrugated web.

Koenen Floors.—Slabs are haunched at each side near the supporting piers or joists, the reinforcement of round bars lying flat between the haunches and being turned up at the ends so as to be securely anchored.

Lindsay Floors.—Slabs may be bounded by girders on two opposite sides, and by rolled steel joists on the other two. Pairs of round bars cross, in the vertical plane in the middle of the slab, one bar passing over, and the other under, the next joist.

Potter Floors.—The tensional reinforcement consists of reinforced corrugated rods. When the span exceeds 12 ft., rolled steel joists are introduced.

Ridley Cammell Floors, Columns, etc.—The beam reinforcement is a trough of dovetail corrugated steel sheeting and angle bars, the slab reinforcement consisting of corrugated sheets. Concrete is applied to both sides of the reinforcement, but centering, etc., is unnecessary, it is claimed. Column reinforcement is a combination of the corrugated sheet and bars of various sections to form a cage. For walls, also, the sheeting is used, studs, if required, being provided by bars.

Somerville Floors.—Reinforced flat-bottomed hollow blocks with curved tops are supported by the flanges of rolled steel joists, a top layer of concrete completing

the floor. In another type, the blocks, much as before, are supported by reinforced concrete beams, which, when the upper layer of concrete is in place, become tee-beams.

Wells Beams, Floors, etc.—The tensional reinforcement of a beam consists of twin bars connected by a web, the bars being placed on edge when several are employed. Some of the bars may be bent up at the ends, and, in addition, there may be straight bars to take part of the compression. The shear reinforcement consists of vertical stirrups, these being known as "hangers" when they connect the upper and lower reinforcements, and as "bonders" when projecting downwards from the compression bars to the neutral axis. Slabs connect main and secondary beams in the usual way to form floors. Rectangular columns of moderate size have a round bar near each angle, the transverse ties being round links with a hook-and-eye joint. In larger columns, the links are connected to the vertical reinforcements.

Wilkinson Floors.—Floor slabs have round bar reinforcements in both directions, with their ends bent over for more secure anchorage.

Williams System.—Small beams are reinforced near the lower surface with I-section bars which take the tension, while the shear is taken by vertical round bars split at the ends. Larger beams have I-section compression bars, and still heavier ones are reinforced with railway metals, and with diagonal bars so attached to the reinforcements that the whole is self-supporting in the mould. The slabs connecting floor beams contain small rolled joists placed parallel and laced together with hoop steel passing above and below the small joists alternately. Joists with flat steel bars riveted to them are used in piles constructed on the Williams system, the end of the joist being shaped to act as a driving point.

The Architectural and Surface Treatment of Reinforced Concrete

Although European and American architects are now using reinforced concrete extensively, it must be admitted that, as far as buildings of any architectural pretension are concerned, it is almost entirely used in hidden situations, such as foundations, floors, beams, columns, etc., or perhaps as a backing to the external walling, either brickwork or masonry, and its use rarely modifies the external expression of the building, except in the case perhaps of commercial or industrial architecture. As an example of reinforced concrete building clothed by masonry, attention may be directed to Mr. W. Aubrey Thomas's Royal Liver Building, Liverpool (*see* pages 329 to 333), in which the external walls are of granite not more than 14 in. thick. In this building the great difficulties, from an architectural point of view, of dealing with an erection of its height have been very successfully overcome.

The statement is frequently made that architects do not like reinforced concrete, and to a certain extent this is true. Why? Probably the reason is to be found in the fact that concrete has little or no inherent beauty. Building stones, slates, bricks, and tiles have a beauty of their own, either of colour or of texture, or both, irrespective altogether of the design of which they form a part, and this beauty is generally increased by the action of the weather. This is also true of timber, with its infinite variation of graining and colour. This being so, architects have not in the past considered reinforced concrete as likely to become a serious competitor with traditional materials for external walls.

Many who visited the Paris 1900 Exhibition probably had reinforced concrete brought to their notice for the first time. This exhibition had undoubtedly much influence on the recognition of reinforced concrete in England and throughout Europe generally, and later exhibitions have still further advertised its claims. The larger class of exhibition buildings are admirably adapted to advance the architectural claims of the "new" material, as in their design a degree of freedom may be permissible that would not be allowed in more enduring erections. This has been, to some extent, taken advantage of, but not so largely as might be wished.

In considering the question of reinforced concrete and the influence it is likely to have on architectural design in the future, it must be admitted that a really satisfactory treatment typical of the material has yet to be found. This will not be achieved by any one man, but will only be brought about gradually, in the course of time, as the material comes into more general use in architecture, each architect adding his small contribution towards the solution of the problem. In other words, it will be a growth, just as all past developments have been.

The guiding principle of the civil engineer is to use as little material as possible, and this explains why he has so readily adopted reinforced concrete, for that is precisely the principle on which it is designed. Whether it is an altogether good principle from an architectural point of view is another matter, since the architect in all monumental work uses mass as an element of expression, but it must be remembered that during the evolution of Gothic architecture the loads were carried on comparatively small supports, and these were reduced more and more as the style progressed, the reason being most probably purely æsthetic. A similar tendency to reduce the amount of material employed to a minimum is seen in reinforced concrete design; but in this case it is for frankly economical reasons. Instead of continuous walling, the weight of the building is carried down to the foundations on piers, external and internal, the floors resting on internal beams or on wall lintels spanning from pier to pier, and the exterior skeleton being united by thin panel walls which carry only their own weight and are used merely to keep out the weather. This thinness of the external walling is one of the drawbacks of reinforced concrete from an architectural standpoint, as, owing to the

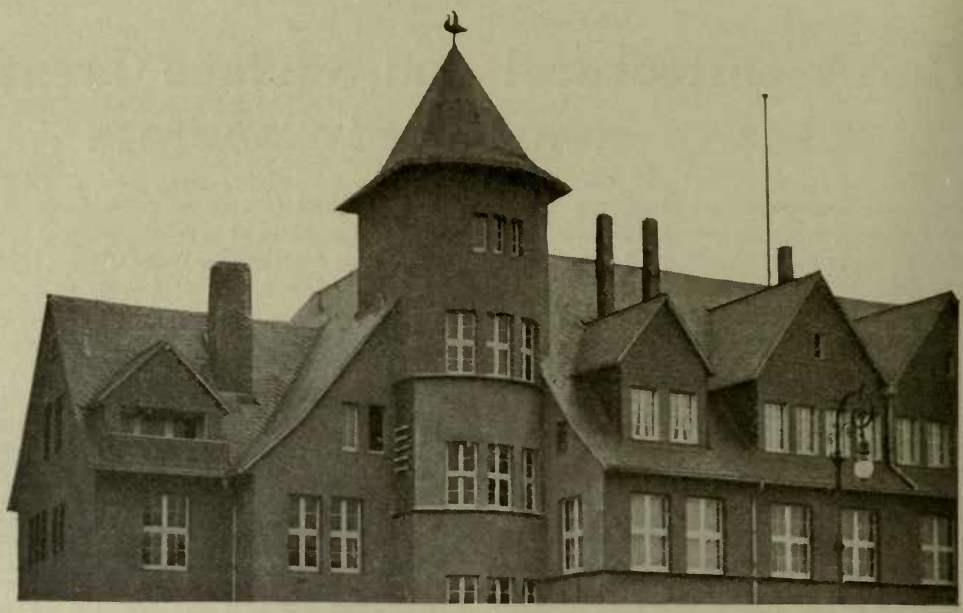


Fig. 424.—The Upper Stories of a Reinforced Concrete Warehouse at Cologne. Front View

Fig. 425.—End View of Reinforced Concrete Warehouse at Cologne

difficulty of obtaining effective shadows, a flat treatment seems almost inevitable, it being impossible to get any depth to the window reveals save by subsidiary piers or other similar expedients. For this and other reasons it does not appear likely that the traditional materials need fear any rival for some long time to come in the largest and most important architectural work. It seems probable that in work of this class its greatest opportunities of affecting design will be in the construction of domes, vaulting, ceiling lights, roofs, and similar works, for which the material is so admirably suited that it is hard to conceive any other method of building by which such good results could be so easily attained.

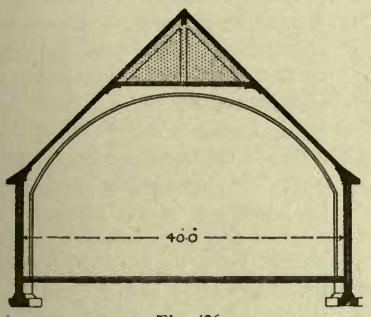


Fig. 426

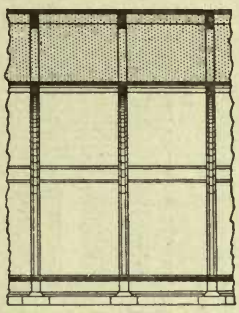


Fig. 428

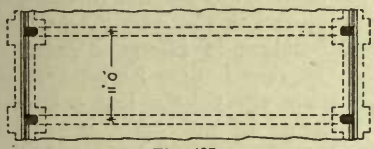


Fig. 427

Figs. 426 to 428.—Vertical and Horizontal Sections of Small Hall with Arched Ribs

Inasmuch as there is a temptation to overrate the position of reinforced concrete and the influence it is likely to have over architectural design in the future, it may be advisable to consider its possibilities and limitations as a constructional material. It is frequently spoken of as though its use enabled larger spaces to be covered than was ever before possible; but this is not the case, and, although we have now in engineering constructions some remarkable examples of the distances that can be spanned by the scientific use of the material, the longest span at present is to be found in the Grafton bridge constructed by the city authorities of Auckland, New Zealand. The bridge is 960 ft. long and 40 ft. wide, and the main arch has a span of 320 ft., although this will be exceeded by the Albert Bells bridge in Rome. It is practically certain that it will always be possible by means of steel to span larger openings than would be the case were reinforced concrete employed. But steel, when used in buildings, must be cased, as, even if its successful treatment from an architectural point of view did not render this necessary, its lack of fire-resisting qualities would make it imperative. This casing, while protecting the steel from fire, is often a source of positive weakness so far as the action of the weather is concerned, as it prevents the periodical painting of the steel. This suggests another great advantage possessed by reinforced concrete, namely, its reduction of maintenance expenses to an absolute minimum. This and the rapidity with which reinforced concrete constructions can be erected are most important factors when comparing the cost of erections by the two methods.

Again, it must be admitted that a reinforced concrete beam is a more truthful, architecturally and scientifically defensible method of construction than a rolled-steel joist cased; and if the former is used in connection with a continuous floor slab the beams are all converted into T-beams, which adds greatly to their strength, the whole being in monolithic connection with the building generally. With reinforced concrete, as with steel, where large spaces have to be

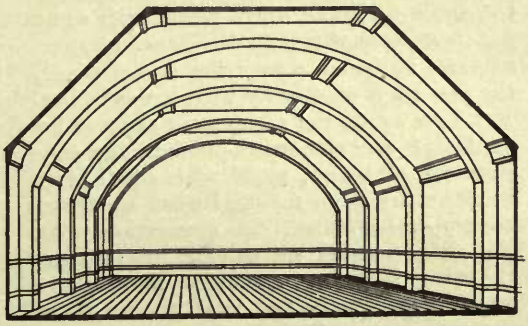


Fig. 429.—Interior View of Hall with Arched Ribs

spanned, depth in the beam is indispensable; but if the architectural arrangements of the interior permit, the beams may be most economically constructed in concrete as arched beams, so reinforced that no thrust is transmitted to the supporting piers.

Some good examples of arched beams carrying very heavy loads are to be found in the Royal Liver building previously mentioned, and in the Wesleyan Central Buildings, Westminster (*see* pp. 337 to 346).

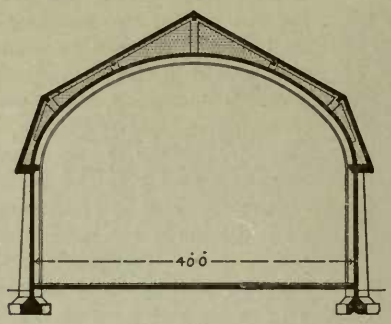


Fig. 430

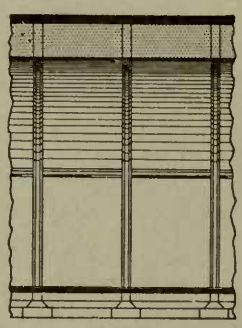


Fig. 432

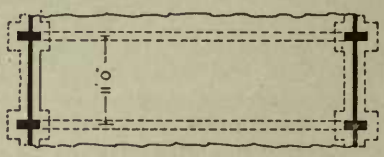


Fig. 431

Figs. 430 to 432.—Vertical and Horizontal Sections of Small Hall with Barrel Roof

With regard to stanchions and columns, it will be found that these, generally, when constructed in reinforced concrete, will work out to about the same or slightly smaller sectional area as would have been the case had a steel stanchion been designed to carry the same load and then cased with some fire-resisting material in the usual manner, so that, as far as proportion is concerned, the position is practically what it was before. Yet, here again, the advantages claimed for reinforced concrete, when used in the construction of beams, could with equal force be claimed for the material when employed in supporting them. In connection with the above remark on column proportions, the two following examples may prove interesting. In the first case, the length of the column was 20 ft. and the load to be carried 200 tons. By designing in reinforced concrete the column worked out to 22⅓ in. square. By selecting a steel stanchion and allowing 2 in. all round for fire casing the result was 18 in. by 17¼ in. In the second case, the length of the column was 12 ft., and the load to be carried 40 tons; the reinforced concrete column worked out to 10 in. square, and the steel stanchion, allowing 2 in. all round as before, to 12 in. by 10 in.

It must not be forgotten that reinforced concrete is introducing no such revolutionary or very great change in architectural expression as that which occurred when the arch superseded the lintel. It is interesting to find Prof. Beresford Pite, in the course of his Carpenters' Hall lecture on "What is Artistic Craftsmanship?" saying: "If the Greeks had had the use of steel girders or ferro-concrete material they would have been delighted to find that they could span the whole of a big front with a beam such as has been employed in the new Post Office building in Newgate Street." It seems practically certain that the Greeks had a knowledge of the arch, and that they preferred and deliberately selected the lintel for covering their openings, in spite of the former rendering larger openings possible but it does not follow that they would not have welcomed a stronger beam, although their desire to span larger openings was not emphatic enough to force them to adopt the arch. Allowing that the physical properties of marble were the main factors that fixed the general proportions of Greek architecture, at any rate as far as the spacing of the columns was concerned, it must be admitted

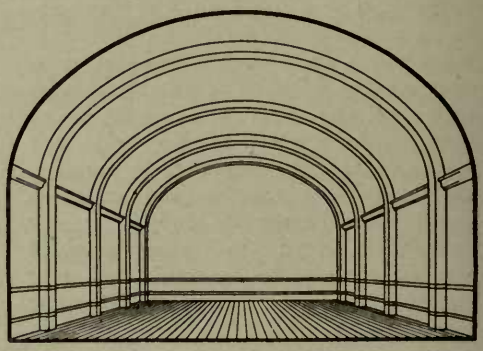


Fig. 433.—Interior View of Hall with Barrel Roof

that no such definite proportions could be expected by using reinforced concrete. A marble or masonry beam of a certain sectional area is capable of doing a definite amount of work; but in the case of reinforced concrete the strength of the beam depends so largely on the reinforcement which is hidden, that no such definiteness is possible, although there is a certain proportion between the concrete and the steel in a beam which may be said to be economically ideal.

The following quotation from Garbett's able book on architectural design is very suggestive: "A new style requires the generalised imitation of Nature and of many previous styles; and a new system requires, in addition to this (as Prof. Whewell has remarked) the binding of all together by a new principle of unity, clearly understood, agreed upon and kept constantly in view. Constructive statics afford three such principles—the depressile, the compressile, and the tensile methods—the beam, the arch, the truss, of which the two former have been made the bases of past systems; the third is ours, to be used in the same manner." In reinforced concrete, the binding principle of the truss is introduced into beams by giving them the rods to take the tensile strain, thus economising material and rendering larger spans possible.

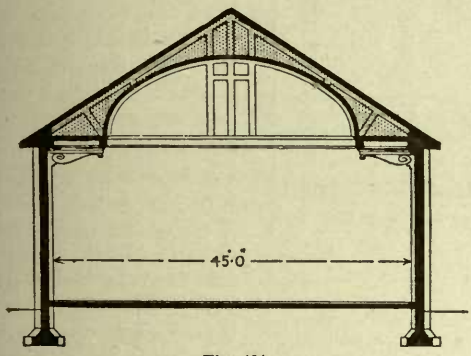


Fig. 434

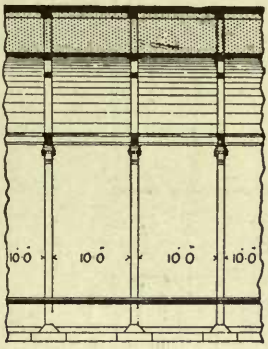


Fig. 436

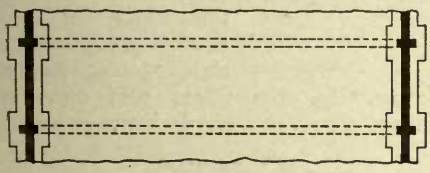


Fig. 435

Figs. 434 to 436.—Vertical and Horizontal Sections of Small Hall with Modified Barrel Roof

It is to some of the larger manufacturing buildings in which reinforced concrete has been employed where the mask walls are uncased, or to engineering works, that we must turn to find treatments which, although guided by economical and strictly constructional reasons, may be said to be most typical of the nature of the material. Obviously, buildings that are to be hidden by masonry are more interesting from a constructional point of view while in the skeleton state than after being clothed. The general resemblance between their proportions and those of steel framework should be noted, but we must avoid letting the material in question have too arbitrary a sway, as we find that from the earliest times, and during the most flourishing periods of architecture, design in one material has frequently been greatly influenced by design in other and sometimes very dissimilar materials. As an example, we have only to take the Greek Doric order, with its triglyphs, guttæ, and mutules, which, although executed in marble, are obviously founded on timber construction.

Taking, then, an ordinary reinforced concrete warehouse, the piers taking the weights will probably be found to show on both the inside and outside; the floors, where they come to the outer wall, will be carried by wall lintels or beams, which also show; the roof will be flat; the exterior mask walls, or panelling, possibly about 5 in. or 6 in. thick, and the window openings fitted with steel casements. The principal defect (leaving out the question of surface finish) is generally the lack of proportion; other defects are flatness and want of shadows. The last mentioned might be partly remedied by keeping the faces of the wall piers, wall lintels, and curtain walls flush, or almost flush, on the inside, and by making the piers of such a shape on plan that they will fit their position in the general design. This can frequently be done without in any way increasing their sectional area. If the main

piers are fairly widely spaced, two subsidiary piers may in some cases be with advantage introduced between them, and if the intervening windows occupy the whole width between the subsidiary piers, more satisfactory reveals can be obtained. The projection of the cornice could generally be greatly increased with advantage to the building. Another way of obtaining good horizontal shadows is by means of balconies, where the character of the building renders them desirable. They can be formed by continuing the floor slab through the wall, the reinforcing rods being continued and turned up in the parapet or balustrade, the whole being in monolithic connection. Where subsidiary piers are inadmissible, and very flat reveals are unavoidable, then probably casements with broad frames round, in whatever material they might be constructed, would give the best results, just as in a flat Georgian treatment we have the broad frames of the sashes with a mould round set nearly flush with the external face of the brickwork.

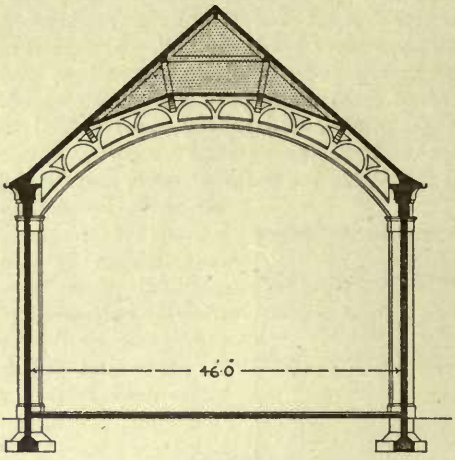


Fig. 437

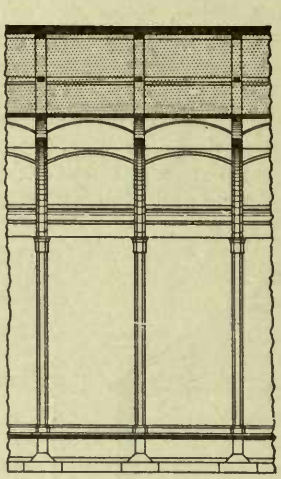


Fig. 439

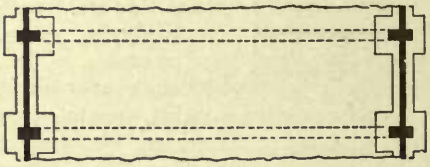


Fig. 438

Figs. 437 to 439.—Vertical and Horizontal Sections of Small Hall with Pierced Arched Ribs

There is an interesting example of reinforced concrete construction on the left bank of the Rhine at Cologne; it is a warehouse, two views of which are shown by Figs. 424 and 425, p. 230. In plan the building is rectangular, having small projections at each end of the main block. The work has been carried out entirely in reinforced concrete subsequently treated with yellow roughcast, which, being subdued in tone, gives a really pleasing colour effect. For the roof rough-dressed local slates of a green tint have been employed in an attractive manner. The general massing of the building is good, and the effect of breaking up the roof surfaces is excellent. It is in the massing of the roof that the chief charm of the building lies; the bold lines of the gables and slate-hung dormers are effective, and the way in which the roof surfaces rise one above the other gives a charm to the design. With regard to the method in which the slates have been used there is much to learn from the Germans, who are apparently far more skilled in the use of slates than are English workmen. In the whole roof a visitor could not discover a single instance of the use of lead. All hips, valleys, gutters, etc., are worked with the slates themselves. This

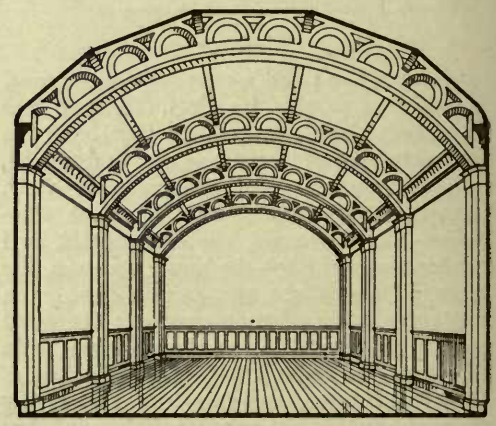


Fig. 440.—Interior View of Hall with Pierced Arched Roof

masterly use of the slates, together with the well-designed sprocketing, has the effect of softening down the roof lines to a remarkable degree. Taken as a whole, the design

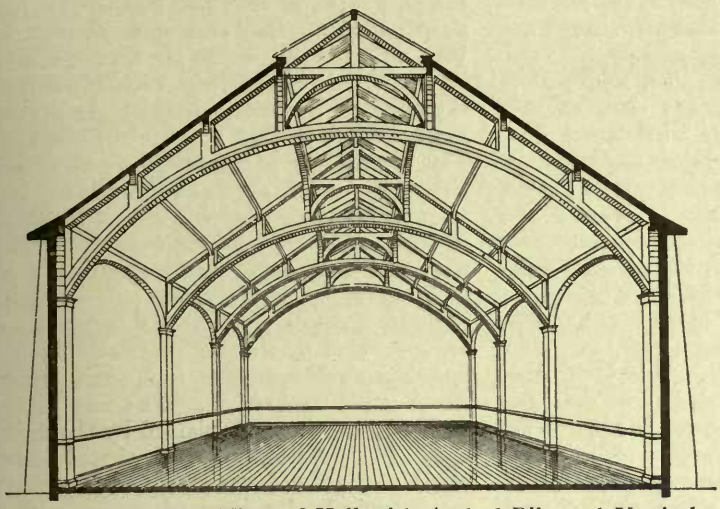


Fig. 441.—Interior View of Hall with Arched Ribs and Vertical Columns

is most instructive, and helps to show what vast possibilities there are for fine architectural treatment of factory buildings, warehouses, etc., in reinforced concrete.

Turning to engineering works, particularly bridges, some very successful and architecturally suggestive designs have been executed in reinforced concrete. In bridges that are typical of reinforced concrete bridge design, there are two arched ribs, or, if the bridge is wide, a series of arched ribs, with vertical columns resting on them, giving the effect of open spandrels, and supporting longitudinal and transverse beams, which in turn carry the road decking. Some very light and graceful effects are obtained by working on these lines, and the treatment also affords suggestions which should prove useful in roof designs.

In the designs which will now be referred to—designs that have been prepared with a view to their suitability for execution in reinforced concrete—the endeavour has been to show how some of the suggestions previously made might be allowed to have their influence; but, of course, the illustrations must be taken as merely showing proposed treatments, and not as working drawings. Figs. 426 to 429 show a cross section, plan, the longitudinal section of two bays, and perspective outline through a building that would be suitable for a small parish hall. The springing being so low, the only form of wooden principal that could have been used would have been of the hammer-beam type, and, the span being 40 ft., this would have made heavy buttresses a necessity. The proposal is to insert reinforced concrete arched ribs, carried down right on to the foundations—the pier being a feature inside as well as outside—and designed so as to exert no outward thrust, connected by continuous reinforced concrete roof and ceiling slabs, the former of which come down on to reinforced concrete beams between the arched ribs at eaves level, and all being in monolithic connection. Below the eaves the outer walls, which are merely mask walls and carry no weight, could be constructed as desired according to the character of the external design and the funds available. No reinforced concrete purlins have been

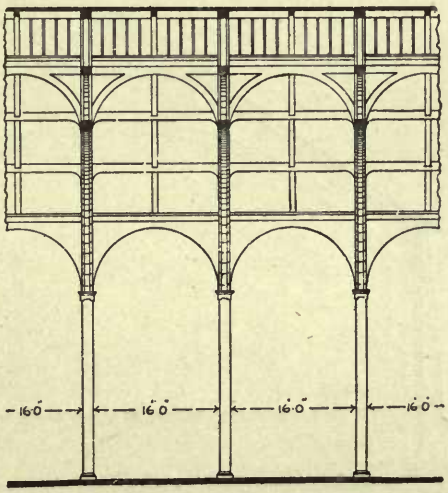


Fig. 442.—Part Longitudinal Section of Hall with Arched Ribs and Vertical Columns

shown, as the idea was by getting away from the effect of timber construction to produce something more typical of the new material; but if inserted they would, by making a

thinner roof slab possible, cheapen the cost. Again, if the arched ribs are looked upon as only taking the place of timber roof trusses, the rest of the roof could be finished with wooden purlins, rafters, etc., in the ordinary way, and would be still cheaper, though not nearly so satisfactory.

By means of sections, plan, and outline perspective, Figs. 430 to 433 show another small hall, but this time with barrel roof, the arched ribs showing below same, springing from piers and being connected at eaves level by reinforced concrete beams, and having continuous reinforced concrete roof slabbing and barrel vault, the external walls only keeping out the weather as before.

The plan and sections in Figs. 434 to 436 show a variation of a barrel roof, with flat, panelled ceilings on each side. In this design the whole is intended to be executed in reinforced concrete, except the mask walls, which are in brick. The arched rib principle is abandoned, and the tie-beam adopted, and although this represents a type well adapted for construction in reinforced concrete, it yet does not seem so typical of the material as the others, but rather more suggestive of timber.

Figs. 437 to 440, with two sections, plan, and perspective sketch, show a rather more elaborate roof, with pierced arched ribs connected by reinforced concrete beams at eaves level, moulded to form interior and exterior frieze and cornice, with roof and ceiling slabs, etc., all as before; but, in addition, the pierced ribs are connected by arched purlins, and the piers inside and out have caps and bases.

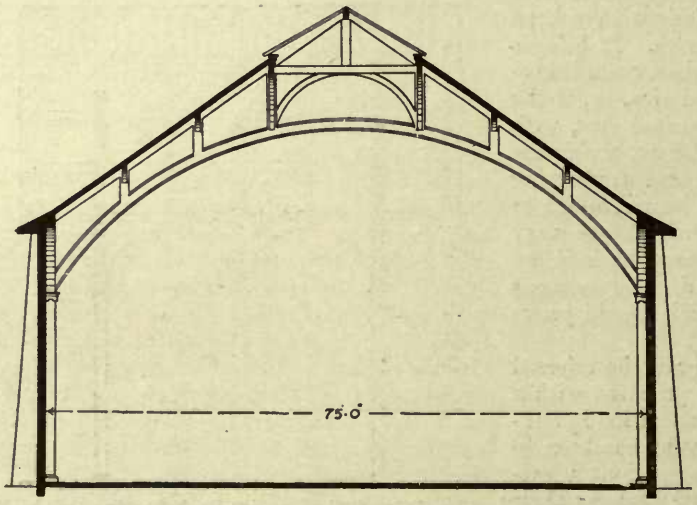


Fig. 443.—Cross Section of Hall with Arched Ribs and Vertical Columns

A type of roof founded on reinforced bridge design is shown in perspective, part longitudinal section, and cross section, by Figs. 441 to 443. There are arched ribs at intervals, as in bridges, with vertical columns, supporting longitudinal and transverse beams, which in turn support the roof slab. This corresponds to the bridge decking, save that it is on the slope, and is treated just as a reinforced concrete floor would be treated, with main beams over the arched ribs, and secondary beams spanning from main beam to main beam. A continuous skylight is shown; the beams supporting it are connected by a series of arched ribs with open spandrels springing from main arched rib to main arched rib. The rest of the design is sufficiently explained by the drawings. This represents a form of roof design capable of great development, and perhaps more typical of reinforced concrete than any of the others.

Figs. 444 to 448 show a design for an exhibition hall, which is illustrated by means of a longitudinal section showing two bays out of three, plan of same, outline perspective, cross section, and part plan of ceiling. It was suggested by the Renommée Hall, Liége, designed by M. Paul Jaspar. As shown, the main hall is 45 ft. wide, with side aisles and galleries all round, 16 ft. 6 in. wide. The main hall is divided into three bays in the length, each being 45 ft. square and having a flat saucer dome, with ceiling light continuing the same curve, over. The saucer domes are of such a radius that if the curve is continued the diameter of the complete semicircle equals the diagonal of the bays. The main piers are connected longitudinally and transversely under the saucer domes by semicircular arches. The saucer domes are ribbed and panelled, exhibiting the real construction, and the ribs, if continued in the same curve, abut either upon the semicircular arches mentioned or upon the main piers, but below the

level of the gallery ceiling, which is also the level of the flat ceiling of the hall. The panelling between the ribs is omitted; in other words, we have what might be called open pendentives, which give a fan-like effect to the ribs springing out of the main piers and continuing those of and supporting the saucer domes. By looking up between these open ribs we see the flat ceiling of the hall, which is, of course, of very small area, as nearly the whole area of the main hall ceiling is taken up by the domes. This is a treatment which contains many suggestions capable of further development. The whole of the work shown is intended to be executed in reinforced concrete.

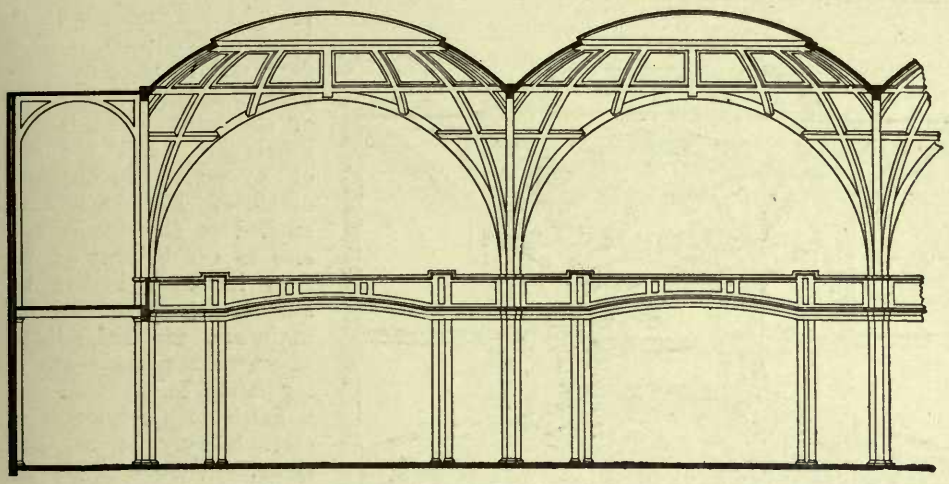


Fig. 444.—Part Longitudinal Section of Exhibition Hall with Three Flat Saucer Domes

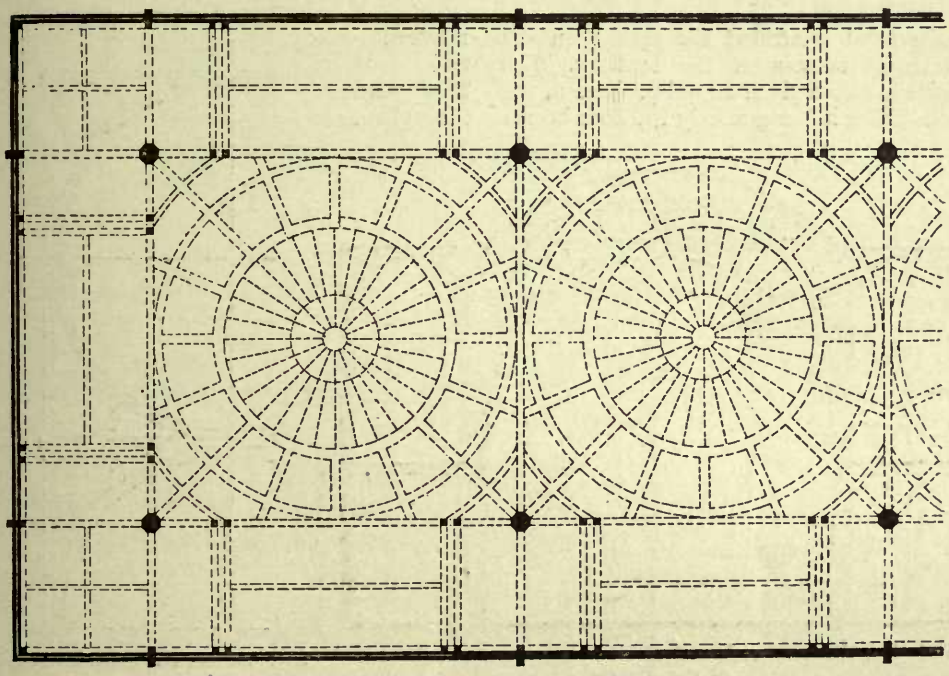


Fig. 445.—Part Plan of Exhibition Hall

A design for a façade suitable for a club or similar building, to be erected in reinforced concrete, is shown in Figs. 449 to 452. The desire has been to produce a design as typical of the material as possible, and to base the architectural treatment throughout on the structural features of the building. The cornice has been given a good projection, the blocks under it being merely the floor beams continued through the wall, with brackets under. The necessity for the blocking being at the back of the cornice to weigh it down, as in masonry construction, having disappeared, it has been brought forward, and the reinforcing rods in the flat roof turned up into it as shown, the whole being in monolithic connection. In order to obtain the effect of a frieze to the building, the filling-in walls to the top story have been brought out flush with the front of the piers, and the floor slab at third-floor level being brought beyond the face of the piers while the wall lintels are flush, the effect of a narrow architrave is obtained. The remarks that applied to the cornice apply also to the balcony at first-floor level. This has been shown with a panelled front in the same material, although, of course, a wrought-iron railing would be just as suitable a finish to a reinforced concrete balcony as to one in stone. In order to get some effective vertical shadows, the front has been treated with a feature at each end and a recessed central portion. The windows throughout are steel casements; those to second floor, having no subsidiary piers to give depth to the reveals, have been given broad frames which lap over the concrete. The wide ground-floor windows to the end features are kept

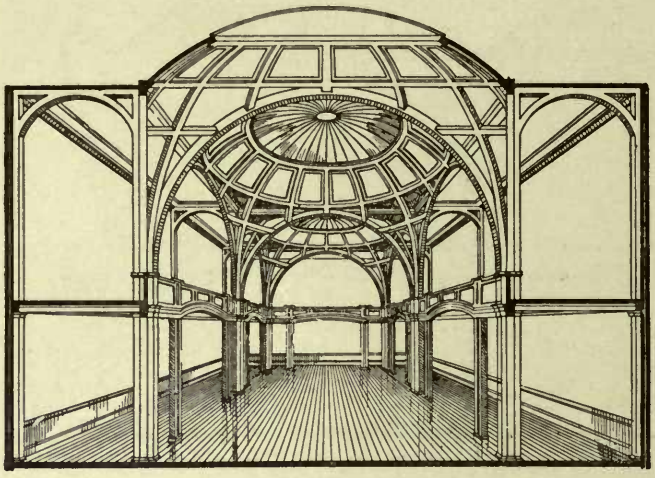


Fig. 446.—Interior View of Exhibition Hall

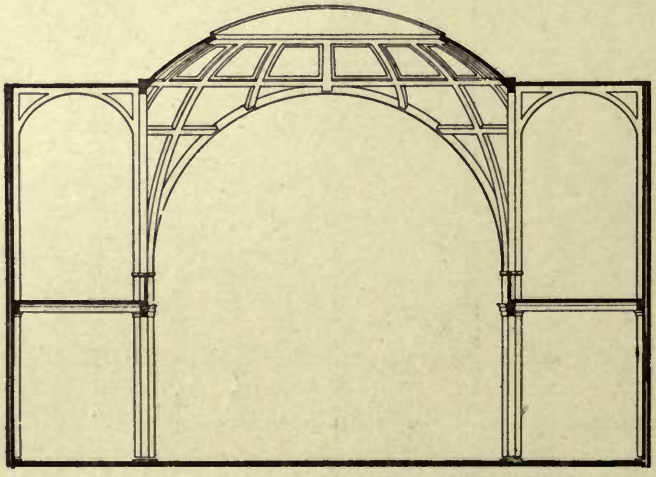


Fig. 447.—Cross Section of Exhibition Hall through one of the Domes

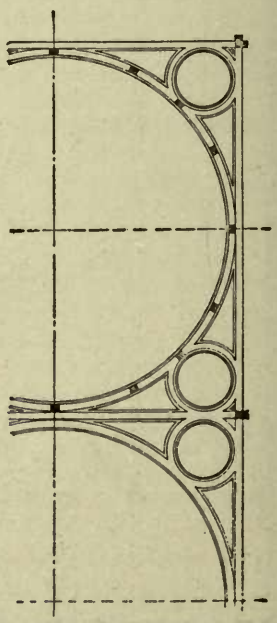


Fig. 448.—Part Plan of Ceiling of Exhibition Hall

nearly flush with the back of the piers, so that good reveals are obtained. The rest of the ground-floor windows are also kept back, but small subsidiary piers are introduced, and the filling in is on the cant. The first- and second-floor windows to the end features are treated as flat bays, the filling in sloping from the small subsidiary piers to the back of the main piers.

The remaining first-floor windows should have small subsidiary piers, which are treated externally as pilasters, with brackets and hoods ovelr. The hoods are brought out in a curve from the face of the wall, as is often done in roughcast, this seeming to suggest a plastic material.

The filling-in, or mask, walls have been designed so as to suggest panelling, and the face of the wall lintels to the second floor has been kept back from the face of the piers and flush with the filling, so as to give more pleasing proportions.

The exterior surface is intended to be finished without any applied ornament. The exterior of piers and beams would be finished with a smooth surface, obtained by

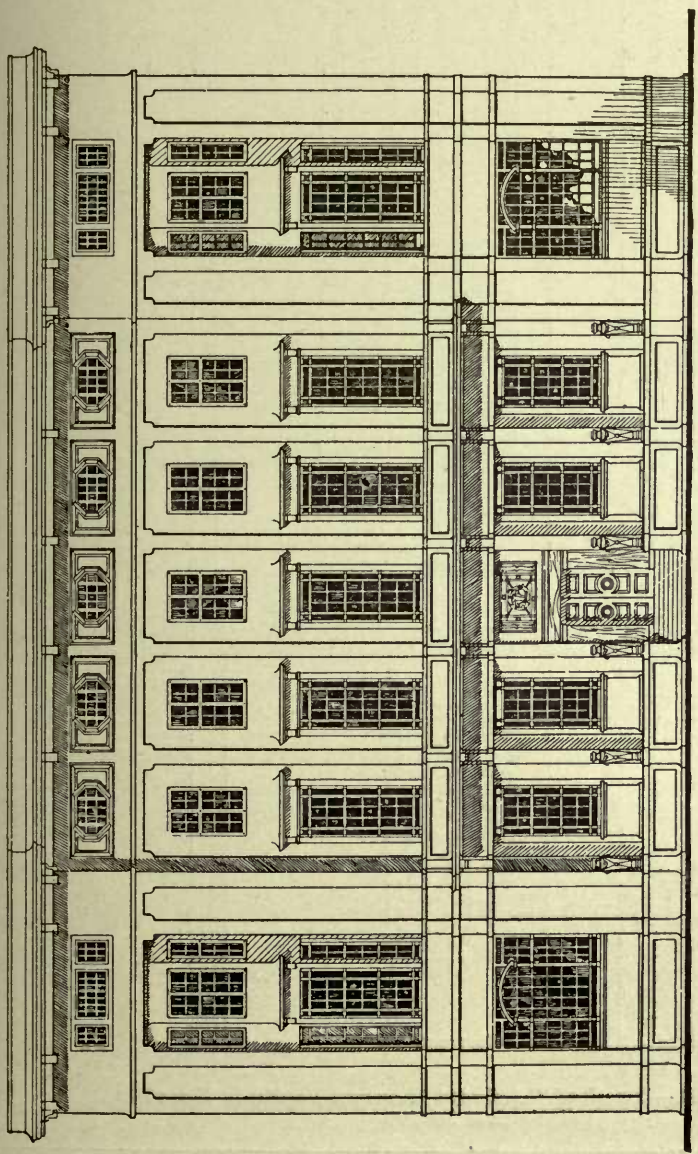


Fig. 449

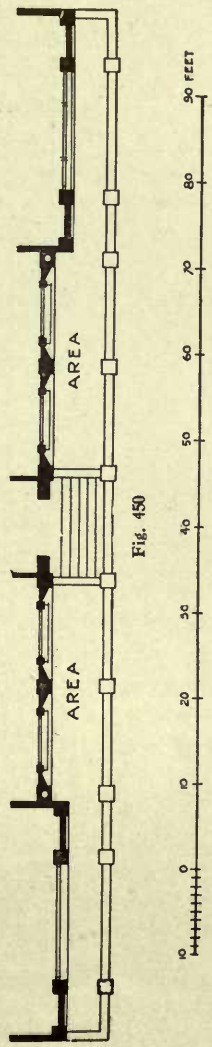


Fig. 450

Figs. 449 and 450.—Elevation and Horizontal Section of Reinforced Concrete Façade suitable for Club

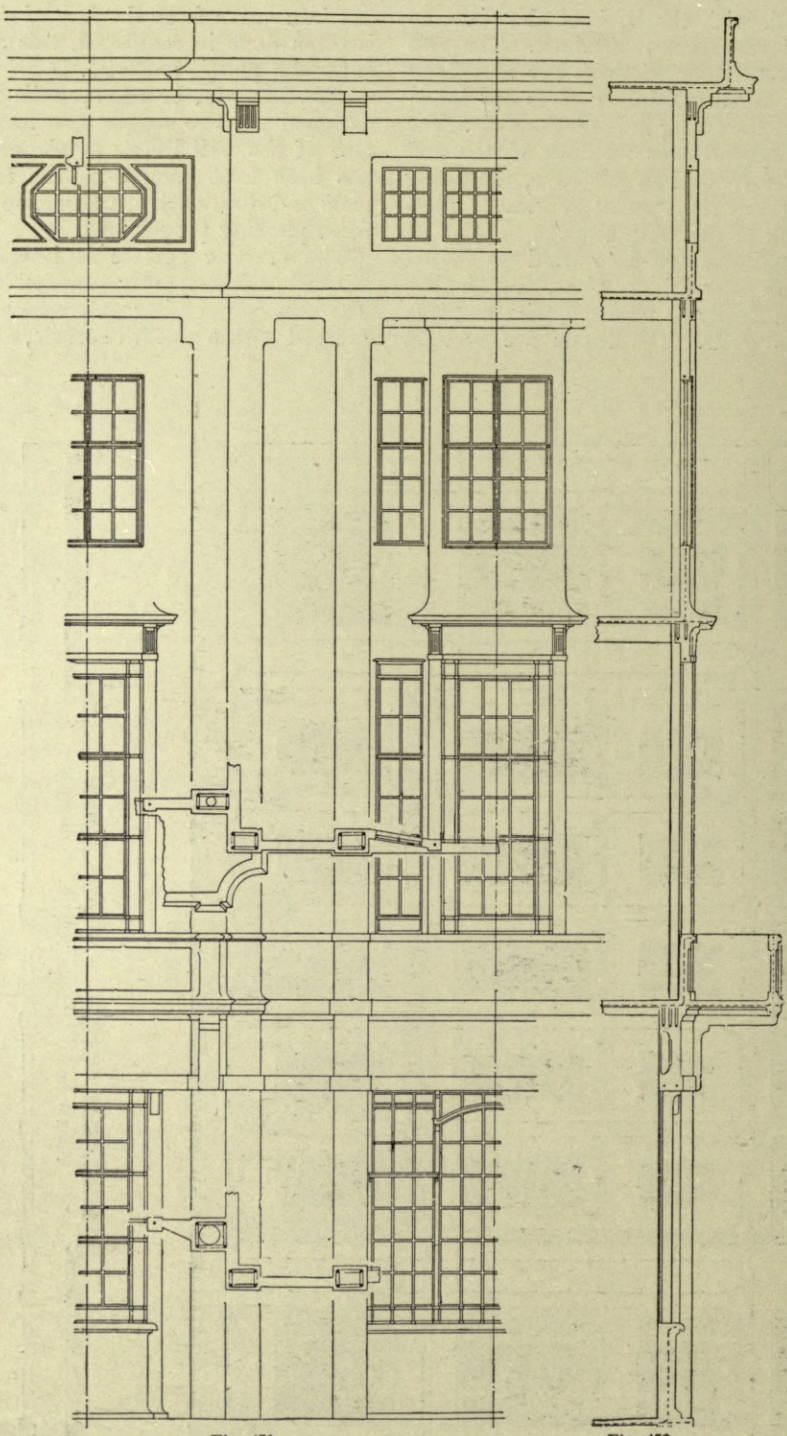


Fig. 451 Fig. 452

Figs. 451 and 452.—Detail of Reinforced Concrete Façade in Elevation and Vertical Section

rubbing down the surfaces after the removal of the moulds and floating them with a thin wash of cement and sand grout. The exterior of panels and other sunk surfaces would be treated by washing out the outer film of cement so as to expose the particles of stone used as matrix and aggregate.

As forming a contrast to the illustrations above referred to, attention will now be directed to Figs. 453 to 457, which, while scarcely typifying, undoubtedly suggest the place taken in the past by reinforced concrete considered purely architecturally. The illustrations show a small public hall or parish hall apparently built of traditional materials, the exterior walls being in local bricks with red facings, and the interior walls being in Fletton bricks. These walls are merely mask walls, however, carrying no weight, and they could just as well, apart from æsthetic reasons, be of reinforced concrete 6 in. thick, finished, say, with roughcast. The roof and the principals are of reinforced concrete, the construction being sufficiently shown by the cross section, a feature in which is the arched beam whose virtues have already been briefly noted.

Fig. 453.—Small Hall with Mask Walls of Brick and Principals and Roof of Reinforced Concrete

SURFACE TREATMENT

The reason offered by architects why reinforced concrete is nearly always covered up is its alleged lack of inherent beauty as a building material. Fortunately, the general adoption, some time in the future, of more artistic methods of finishing will remove the reproach. Clearly, until reinforced concrete is boldly exposed it will never be able to exert its legitimate influence over external design, although, even as now used, it has had a certain influence on design, particularly in commercial architecture. Let it be remembered that a building covered with stone will always be described by the general public as a stone building, of whatever material the backing may consist.

The question, therefore, of the surface treatment of concrete is one of importance to the future of reinforced concrete, especially in view of the fact that the repeal of restrictive legislation now permits thin exterior walls (in other words, mask walls) to be employed. The subject of surface treatment has already received great attention, but the matter is one for further experiment; and in the search for more pleasing finishes—those whose effect

will be enhanced, not ruined, by the action of the weather—the knowledge of the chemist, as well as of the practical man, should be enlisted. Definite examples of the results obtained, as well as of the methods ordinarily adopted, should be easily accessible for inspection by architects generally.

The methods of finishing concrete surfaces will now be considered in detail. They may be classified under two main headings: (1) natural finishes, and (2) applied finishes.

By "natural finishes" are meant those obtainable simply by varying the aggregate and the nature of the facing in the form, the surface left on removing the false-work undergoing either no treatment at all or only sufficient to bring into relief the aggregate in the facing or to make the texture uniform.

"Applied finishes" are those obtained by the application of plastic materials.

The Untouched Surface.—A common finish, but one that æsthetically fails to

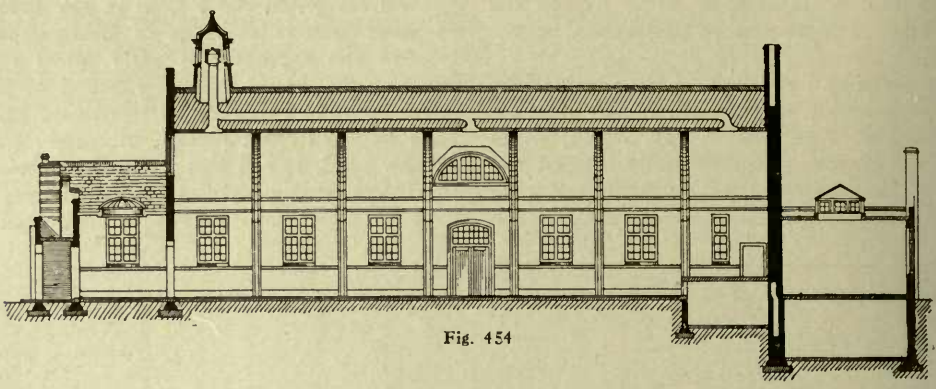


Fig. 454

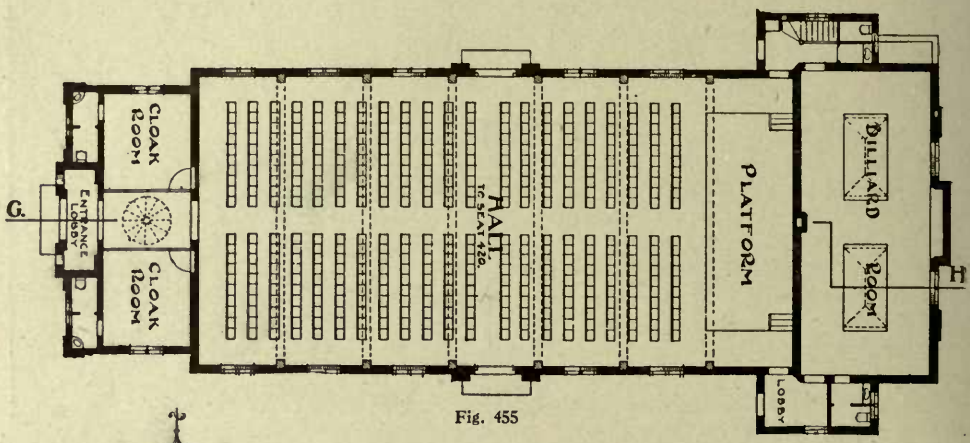


Fig. 455

Fig. 456

Figs. 454 to 457.—Longitudinal Section, Plan, Front Elevation and Cross Section of Hall with Reinforced Concrete Principals and Roof

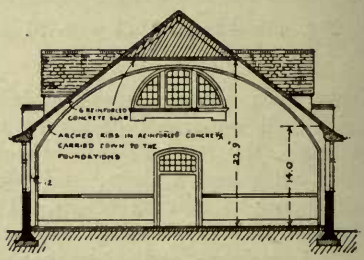


Fig. 457

please many people, is to leave the wall just as it comes from the forms, with all the board marks showing; but what it lacks in beauty it makes up in efficiency. C. R. Knapp, in a paper read before the National Association of Cement Users (U.S.A.), mentioned an instance in which the original cement skin had been left undisturbed, and in which it defied for a number of years the penetrative power of water which lay in a pool over a ceiling. He stated that after protracted storms the surface seems to dry off instantly, while neighbouring houses of brick, and even frame construction, retain evidences of moisture upon the outside long after the concrete house has assumed its natural colour. Not only, said Mr. Knapp, was £40 to £60 saved for tool dressing, but, in the opinion of the architect, the finish was more artistic, as well as indicative of the plastic nature of concrete.

It may be taken as granted, however, in spite of an occasional opinion to the contrary, that the uninteresting nature of the undisturbed finish, its drabness, and its entire lack of light and shade effects, are obstacles to the employment of concrete in many quarters, and that something different will nearly always be demanded.

The Brush Finish.—One of the cheapest and most satisfactory methods of treating the green concrete is to give it a stiff brushing, taking care to make the effect as uniform as possible, it being quite easy, either with an ordinary scrubbing brush or with a wire brush, to vary the "pattern" as the work proceeds.

It will be noted that the concrete must be green; but it is not possible to give a definite time from the placing of the concrete to the commencement of the brushing, as naturally it will depend upon the proportions of the ingredients, the state of the weather, etc. Should the brushing be attempted before the concrete is sufficiently set, the result will be irregular, as any small aggregate present may easily be broken out; on the other hand, should the brushing be delayed too long, the concrete may have become too hard to be treated by this method.

The domestic scrubbing brush answers when the concrete is very green, and the wire brush when the concrete is harder. It is customary to flow water over the surface of the concrete freely during the course of the brushing, and this is undoubtedly most conveniently done by means of a hose with a rose head.

It is customary to assist the brushing by treating the washed surface with dilute muriatic or acetic acid, which helps to clean the aggregate and conduces to uniformity of texture throughout the whole job, which very possibly has been scrubbed down at varying intervals of time. One part of acid diluted with three parts of clean water is suitable; but when the concrete has been made with white cement and white stone aggregate, it is better to use a 1 : 3 dilute sulphuric acid.

An objection to the use of acids (muriatic, acetic, etc.) is the possibility of the formation of stains and efflorescence, which certainly are very unsightly. An acid-treated surface needs to be well washed with plenty of water.

The Carborundum Finish.—The well-known carborundum finish gives a light-coloured surface and, by filling in the pores with cement, renders the material less pervious to water. Immediately the forms are removed, the surface is wetted and then rubbed with a No. 16 carborundum stone until a lather forms and extreme roughness has been removed. The work is washed down with the help of a brush, and while wet dusted with a 1 : 2 mixture of cement and fine sand, which is next rubbed in with a No. 16 stone, the finish being applied by rubbing with a No. 30 stone. A committee appointed by the National Association of Cement Users (U.S.A.) has recommended this method, but specified the use in the early stages of a No. 8 carborundum stone brick instead of the (finer) No. 16.

The Sand-blast Finish.—Sand-blasting has proved a quick and convenient method of finishing, the resultant surface rather resembling that produced by scrubbing, but being more regular. The principle is the abrasion of the surface by particles of sharp sand carried at a high velocity by a current of air under pressure. The abrasive removes the board marks and produces a uniform matt finish.

Whilst practically any type of sand-blast apparatus can be adapted to the purpose, there is much to be said for the Niagara pattern illustrated by Fig. 458. This is a simple T-piece weighing about 6 lb., and it acts on a well-known principle exemplified by the familiar scent-sprayer. The air, under pressure of from 80 lb. to 90 lb. per

square inch, is supplied through the flexible $1\frac{1}{4}$-in. hose shown, and, in popular language, produces suction in the $1\frac{1}{4}$-in. vertical pipe, whose lower end, weighted, is placed in the

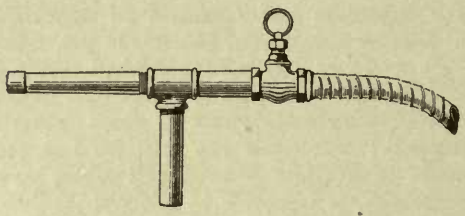


Fig. 458.—T-piece for Sand-blast Apparatus

sand. The sand, by the way, can be held in a vessel or may simply be in a heap. When the device needs cleaning, the nozzle is pressed against a hard surface, this causing the compressed air to find an outlet through the suction pipe and effectually clearing this of any obstruction. Obviously, any similar material to sand can be used with the device, and it may sometimes be an advantage to use something with less pronounced cutting properties.

Bush-hammered Finish. — What is known as "bush hammering" — so-called from the use of the mason's bush hammer, one form of which tool is illustrated by Fig. 459—destroys the facing film of cement, and therefore injuriously affects the water-tightness of the work; but where this is unimportant, the method can be recommended as producing an interesting surface, the cement between the aggregate being chipped out and the stone aggregate itself being roughened. The work should be done within

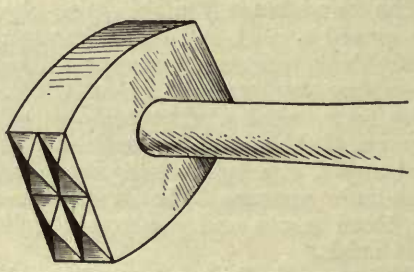


Fig. 459.—A Type of Bush Hammer

about two months of setting, as otherwise the concrete may be too hard to be affected by the tool. Stone masons' tools of the axe and patent-axe type will be familiar to most readers; the bush hammer is used in practically the same way, either by hand or in a compressed-air machine tool.

Facing Concrete in the Form.—The full advantage of most of the above methods of finishing can only be realised when special precautions have been taken to see that the work is given a facing—at least 1 in. thick—of fine material in the form; an applied facing, plastered on the hard concrete, is not the same thing. The facing is applied in course of executing the main concreting by plastering it on the form and then introducing the body concrete; or, instead, the body concrete is inserted and then pushed back from the form to allow of the fine stuff being introduced; still another method is to use iron plates, as described later. By "fine stuff" is meant a mortar or concrete with a small aggregate suitable to show on the face of the work; it certainly does not mean excessively rich stuff, as this is always liable to go "crazy," that is, develop hair cracks all over its surface. 1 of cement to from 2 to 3 of fine aggregate is a suitable proportion. When the facing is to contain both fine and coarse aggregates, suitable proportions are $1:1\frac{1}{2}:3$, $1:2:3$, or even $1:2:4$, the ingredients being cement, sand or stone screenings and crushed stone or screened gravel, and the facing must be twice as thick as the diameter of the largest stone in the aggregate.

To facilitate the placing of both the facing mixture and the backing without mutual interference, the use of iron plates—"granolithic plates"—is recommended. These may be of any suitable depth and length, 12 in. × 5 ft. or 6 ft. being suitable, furnished with a handle near each end and flared off at the top, as in Figs. 460 to 462. On the side opposite to the flare two or three angle-irons or T-irons are riveted to act as distance pieces. As many of these plates as required are put into the forms, their ends slightly overlapping, with the angles or T's close to the face of the form, in this way forming a deep, narrow slot into which the facing stuff can easily be poured, the flaring of the plate assisting this. The concrete backing is poured in at the same time, but the facing mixture is kept at a slightly higher level to prevent any thin stuff from the concrete running over the top of the facing and finding its way to the face of the form. When the layer is of the desired thickness, the plates are raised in readiness for the next layer, and the two materials—the body

concrete and the facing—brought into intimate union by ramming.

When the aggregate in the facing material has been selected especially to give character to the concrete, care must be taken to ensure its fineness, and it will be necessary to remove the cement skin from the face of the work. It is usual to use as aggregate stone or pebbles crushed to pass through, say, a $\frac{1}{4}$-in. screen, but to be retained upon a $\frac{1}{2}$-in. screen. An average diameter of $\frac{3}{8}$ in. is about right. The fine stuff is applied to the face of the form by means of a trowel or plasterer's float just before introducing the body concrete; or the "granolithic plate" method already described is employed. Care must be taken to get the two materials into intimate union. By means of a little thought it is possible to choose aggregates of different colours and sizes to produce excellent effects, and a note may here be made of the suitability of selected ground mica for this purpose, this material supplying "life" and sparkle suggesting freshly wrought granite. The next part of the procedure will depend largely upon the nature of the work and the setting power of the cement used. All that can here be said is that the surface of the granolithic concrete must be exposed while the material is still friable, so that an immediate washing with water and a stiff brush will remove the cement film and expose the aggregate, which will now appear in decided relief and of a rough, coarse texture. (The illustrations, Figs. 463 to 468, on later pages are full-size photographic reproductions of patterns obtained by the above method, and the inscriptions give all necessary particulars. They are due originally to *Engineering News*.) Even when the face has become too hard for brushing, something of the effect can be produced by rubbing with a wooden block, or with sandstone and plenty of water, but this method will not leave the aggregate in relief. Again, bush hammering, as already described, may be adopted. The following suggestions are due to Henry H. Quimby, M.Am.Soc.C.E.: "If the height of the wall to be thus treated is too great to be completed in one day, face forms must be constructed to facilitate the removal of the planking without disturbing the studs or uprights. This is easily accomplished by setting the studs 8 in. to 12 in. away from the face line and supporting planks with cleats—say 2-in. by 1-in.—tacked to the studs and the planks. This permits the lower planks to be removed and the washing done while the upper planks are in place and concrete is being deposited. With the exercise of very watchful care on the part of the workmen and unremitting inspection, two different days' work can be joined so that, after washing, the joint will not be unsightly—even scarcely distinguishable; but such work is usually not obtainable throughout a structure, and it is found very easy to obtain thoroughly satisfactory joints by indenting horizontal grooves at regular intervals representing courses, and finishing each day's work at the apex of a groove. These indentations are made by means of triangular beads on the face forms. Usually the bead is the bevelled edge of a strip set between the face planks and lightly secured to the planks with partly driven toe nails, so that, if desired, a plank can be removed independently of the bead above it, the bead remaining to set the plank upon the next course. These grooves in the face of a wall improve the appearance by relieving the blankness of a large area. It is found practicable to prosecute the work with one course of planks where the capacity of the plant for one day is equal to only one

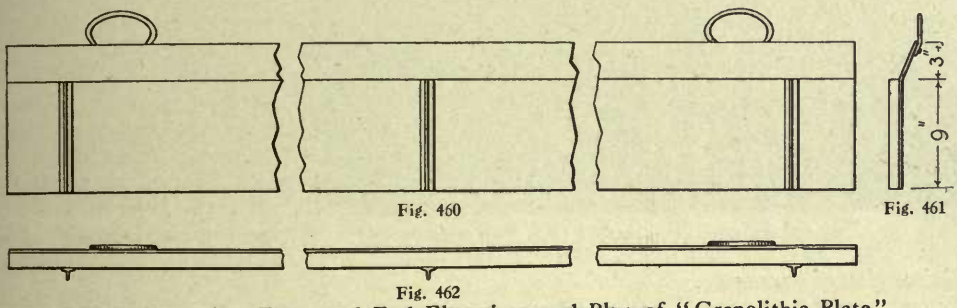


Fig. 460 Fig. 461

Fig. 462

Figs. 460 to 462.—Front and End Elevations and Plan of "Granolithic Plate"

course of concrete. In this way the same planks have been used for many different courses on four or more different structures."

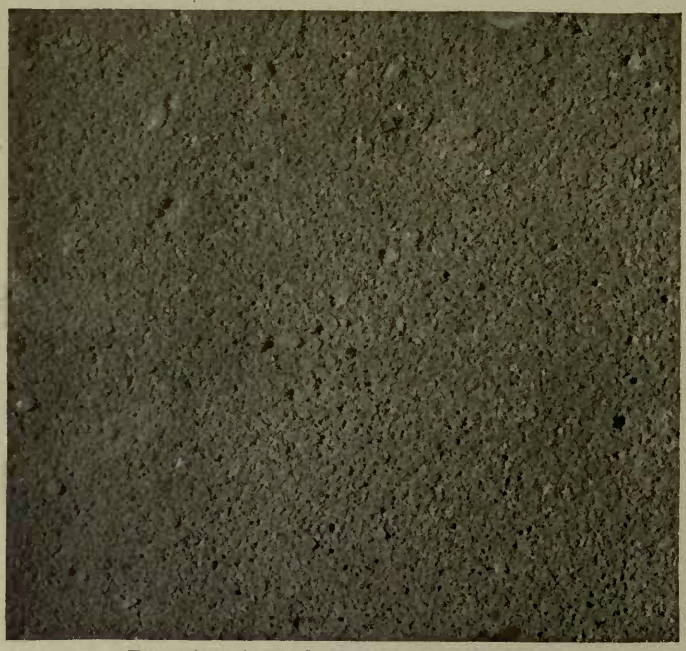


Fig. 463.—Sand Concrete, 1 : 2. Full size

Uniformity of aggregate, both as regards size and colour, is of great importance in this method of finishing. The matter needs careful attention before any work is started in order to minimise the risk of being obliged to complete the walling with aggregate of a different character.

The nature of the tamping exercises a great influence on the texture of the face of the work. For example, when a large aggregate is wished to show in the face of a wall, the tamping should be done in the middle, this carrying the stones against the form. Tamping near the side helps in getting the finer stuff towards the face. In producing horizontal, flat surfaces, skilful tamping will bring the finer stuff to the top, but a special tamper—the "Andrews," of American origin, but obtainable in both the United States and Great Britain—has been introduced to facilitate this particular work. As shown on p. 129, the tamper has a number of pyramidal points on its working surface, so arranged that when a mass of concrete containing large particles of stone and gravel is tamped, these larger particles are pushed down further into the mass, and the thin stuff flows to the surface. The tool is 8 in. square, and the points projecting from the face are connected at their bases in such a way as to prevent particles of stone from becoming jammed between them. In use, the tamper is employed after the layer of concrete has been spread and roughly levelled. After its

Fig. 464.—Crushed Stone Concrete (Cement 1, Yellow Bank Sand 2, and ⅜ in. Screened Stone 3). Full size

use, the surface may be smoothed off and finished like any other finish dressing.

To cause the thin stuff in the wet mixture to flow to the side of the form, and thus obtain a fine vertical facing without introducing a second mixture, the use of a perforated spade (*see* p. 128) has been suggested; and many contractors will find it convenient to carry out experiments with the object of applying the idea in other ways. A spade illustrated on p. 129 is that known as the Ross. Its action is to force back the coarser aggregate and to allow the finer stuff to come to the face of the work. Other tools for a similar purpose are illustrated on pp. 128 and 129.

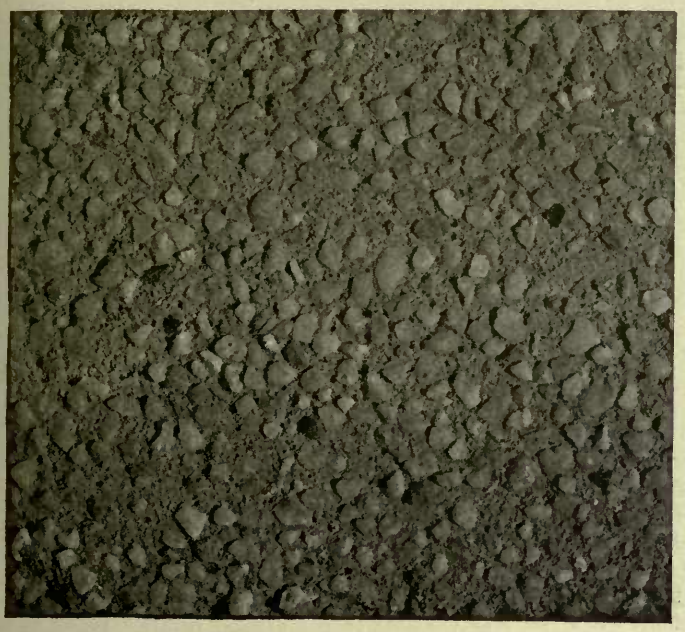


Fig. 465.—Pebble Concrete with Scrubbed Surface (Cement 1, Bar Sand 2, and $\frac{3}{16}$ in. White Pebbles 3). Full size

Pebble Dashing.—To imitate the effect of stucco upon which pebbles have been dashed, the face of the form is plastered with well-worked wet clay into which have been lightly pressed small pebbles, pieces of glass or glazed tile, marble chips, etc. When, after the concrete has set, the form is taken away, the clay is washed off and the surface brushed to reveal the pebbles, etc., in the face of the concrete.

Sand Finish.—The method is on the lines of that described in the preceding paragraph. Clay is well tempered and thoroughly worked, and then plastered on the inside of the forms, working it, if desired, into patterns. Sand is applied evenly to the wet clay, and then the concrete is poured in. On the removal of the forms, the clay is washed off with brush and water, the sand adhering to the concrete. This method certainly appears to be unnecessarily troublesome. Merely to get a sand finish on plain, flat walling, it should suffice to brush over the

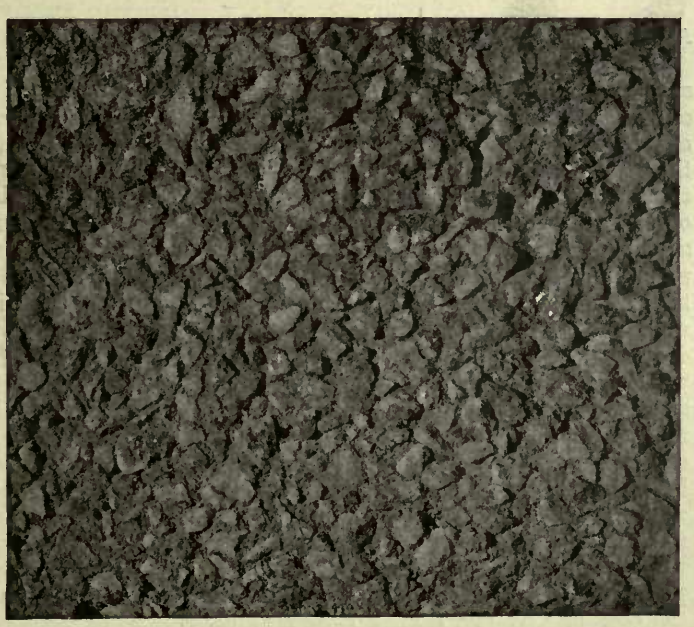


Fig. 466.—Granite Grit Concrete (Cement 1, Bar Sand 2, and $\frac{1}{4}$ in. Granite Grit 3). Full size

form with clay water, and apply the sand, which will readily adhere.

Glazed Finish.—Whilst concrete cannot be truly polished, it is not difficult in theory to obtain on it a glazed surface. All that would be necessary would be to line the form where required with well-cleaned and polished glass, or, as an American experimenter has proposed, with enamelled iron plates, of the kind universally employed for large advertising signs. In practice, the use of such an untractable and brittle material as glass would cause never-ending difficulty, although it might be possible without much inconvenience to introduce narrow slips for the purpose of casting ornamental devices on the work. The employment of steel forms stove-enamelled on the working side might provide a solution to the problem should the production of glazed surfaces be demanded. A practical difficulty would be the almost inevitable formation on the form of air-bells, as these would, of course, spoil the surface; but the trouble might be got over by first lining the form with fine cement mortar ½ in. or less in thickness, applying it with heavy pressure. It is reasonable to assume that the polished skin would give the concrete more than usual resistance to weather and to mechanical wear.

APPLIED FINISHES

Plastered Surfaces.—When a concrete surface is finished by coating with plaster of whatever nature, there is always a risk of this coming away at a later date should there be a likelihood of moisture percolating through the concrete and affecting the back of the plaster. However, plastering is often successful, but much depends upon the preparation of the concrete surface and the kind of plaster used. A good plan is to wash the concrete with dilute acid, taking care to rinse this off afterwards, and then, while the surface is damp, but not flooded with water, to apply the plaster. The moisture in the surface of the concrete assists the cement mortar to set. The following method, advanced by an American contractor, should give good results. The first coat is a 1 : 3 mixture as dry as it can be applied, this necessitating considerable pressure to cause it to adhere well. Let the coat be thin and scratch it afterwards. Brush over the first coat with water and apply the second coat, using as much pressure as possible. If a third coat is necessary, adopt the same precautions.

A concrete surface gives a better key for plaster after it has been hacked over with axe or bush hammer, but this necessitates a thick coat of plaster. If the concrete is soft enough to be affected by a coarse wire brush, a rub down with that will answer instead of hacking, and the plaster coat can then be thin. The acid treatment and washing, as described on p. 243, should precede the plastering.

Tiles, Mosaic, Sgraffito, etc.—In some situations, tile, or the old Byzantine material, vitrified mosaic, makes a beautiful and suitable finish. In either case, they would have one constructional advantage, especially in high buildings, over brick or masonry casings, in that the weight added by them to the foundations would be comparatively trifling. A concrete wall having a true, flat surface could be prepared for tiling by rubbing down with a carborundum stone and then scratching with a brush—bristle or wire, according to the age of the concrete; the wall should be wetted and the tiles bedded in neat cement, using as small a quantity as possible to get a good result. Much the same instruction applies to the mosaic.

It is somewhat surprising that sgraffito is not more generally adopted as a finish to concrete work where ornamental effects are desired; certainly in some cases it might be very effectively employed. Sgraffito is an Italian word meaning "scratched," and is the name of a very old process consisting in applying to the surface several coats of different colours, these coats being brought to view as required by chipping or scraping. In the simplest form there are but two coats—black for the ground, white for the covering; and the surface to be treated is floated to a uniform face, allowing for the finishing coat, which will be ¼ in. thick. The outline of the design is marked on the face of the floating, as a guide for keying the ¼-in. finishing coat, which should be applied as soon as the colour coat is sufficiently hard. Placing the design drawn on paper in its original position, pounce through the outline as a guide for cutting through to the colour coat, using a worn knife to cut away the superfluous material, which is removed with a spatula; the edges of the work being sloped or inclined according to the light or shade required. For three-coat work the colour coat may be left rough,

providing it is uniform, a dark colour being used as a background. This coat is ruled to within $\frac{3}{8}$ in. of the finished face, the class of work determining the thickness of the various coats. Apply the succeeding coat, and finish as soon as convenient, so that the coats may adhere in one compact mass. Expedition is specially necessary when treating exterior work, as water settling on the incisions of work improperly keyed would cause it to laminate or scale off. For colouring matter, to obtain good black use bone-black—or, for ordinary work, smiths' ashes as an aggregate; for red, Venetian or Indian red; for brown, umber; for yellow, yellow ochre. When a neutral tint is required, a combination of two or more colours may be employed. The depth of the colour should be ascertained by adding a sample of it to a small quantity of the stuff with which it is to be used, the material being allowed full time for setting. For external work, portland cement or Aberthaw lime may be used, in the proportion of 2 parts sand to 1 part cement. Aberthaw lime may also be used for internal work, and so also may Parian cement; but for ordinary purposes selenitic may be used with satisfactory results.

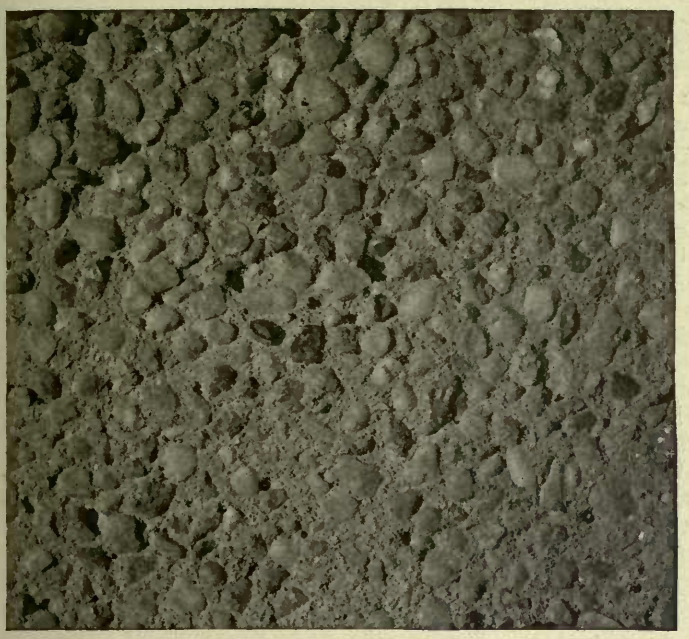


Fig. 467.—Pebble Concrete (Cement 1, Bar Sand 2, and Screened Yellow Pebbles 3). Full size

Fig. 469 (p. 251) shows the excellent effect obtained by the adoption of inlaid faience tiles and red bricks, the roof covering consisting of terra-cotta tiles.

"Stuc" Work.—This is a form of interior treatment particularly suitable for application to concrete walls and lending itself to a simple and effective design. In appearance, it suggests a good Bath stone. The material is made with

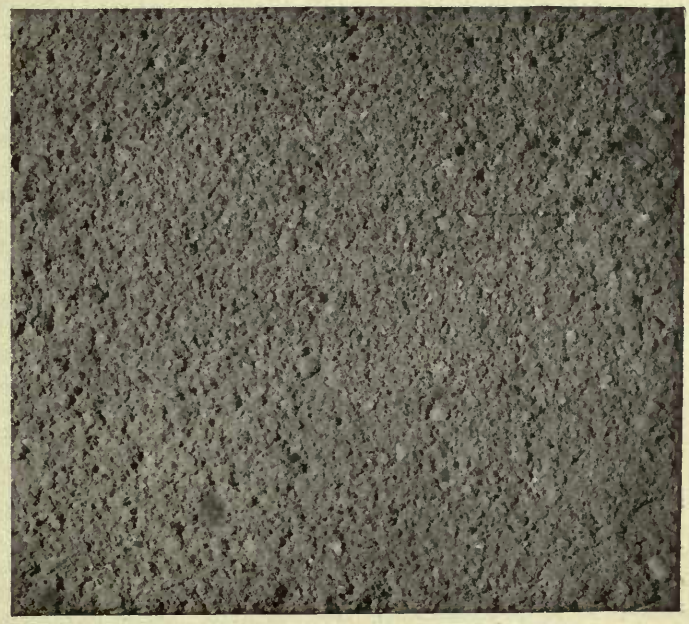


Fig. 468.—Sand Concrete, 1 : 3

a mixture of plaster-of-paris, cement, size and various colouring matters, the latter varying slightly according to the desired finished effect. In the application, a very moist solution of plaster-of-paris is first stippled on the walls with a stiff broom to give a key for the "stuc." The plaster and cement are then placed in a mixing box about 2 ft. by 1 ft. 5 in. and about 12 in. deep, and thoroughly mixed. When this is complete, the size, water and colours are added, and the whole thoroughly incorporated, and the mass is squeezed through the hands until a thick cream is obtained. The resulting compound is very quick in setting, and consequently it requires to be rapidly applied to the surfaces, these having been previously screeded. The hands and a brass trowel are used, the plaster being literally thrown on. Whilst moist, it is roughly lined to the screeds, and the surfaces are planed and scraped to a fairly even and true face. When set, it is left for about two months, and during this period various stains come out to the face of the "stuc," which assumes a light brown colour, and becomes very hard. After this time a scraping plane is employed and the whole of the outer surface is removed, taking away all stains and leaving a fine soft stone effect. During the scraping process, small pit holes are formed in the surface, which greatly improves the appearance, and removes the perfectly smooth and artificial effect otherwise given. The whole surface is then rubbed down with pumicestone, and the various joints are set out with chalked cords in accordance with the design. These lines are then cut in and filled up with white plaster, and a final rubbing down with pumicestone is given. Mouldings and cornices are run in the orthodox manner with a "horse," several plasterers throwing on the material at each point, whilst the horse is worked backwards and forwards.

When using the "stuc" on concrete work it is advisable to give the walls a thin coating of Keene's cement to prevent the staining of the "stuc."

In the execution of the ornament, some excellent work is seen, as this is not cast, but modelled in position by the plasterer, and for this reason every man requires to be an artist fully in sympathy with his work. "Stuc" work was extensively adopted in the decoration of the Royal Automobile Club, and it can be said confidently that the work to the fluted columns round the gallery and the Roman-Doric caps, with the enriched frieze above, are greatly superior to similar work executed in ordinary fibrous plaster. The "stuc" work requires neither painting nor other finish, and retains its fine appearance if merely rubbed down once every three years.

COLOURING CONCRETE

In response to a demand for a brighter and more interesting colour than that provided by natural concrete, there has been a great deal of experiment to determine the best methods by which the desired result can be obtained. Undoubtedly, the most artistic method, but one not suited to all or even the majority of situations, is to determine the colour by the use of a specially selected aggregate which will give, not only colour, but texture to the work. Sufficient has been said on this subject earlier in this chapter. Another method is to face the walls with tiles, mosaic, etc., as already described, but this is expensive, and has the disadvantage of masking the character of the concrete surface, and in that particular sense is false art. Strictly, methods of colouring concrete include only those by which (*a*) pigment is incorporated with the concrete or with the facing material introduced into the form, (*b*) staining or (*c*) distempering, or (*d*) oil-painting the concrete surface.

Body Colours.—But for the fact that the lime in the concrete has an injurious effect upon the majority of pigments, there would be very little to say under this heading. This injurious action has long been known, and everyone is familiar with the bleaching of paint and wallpaper by the lime in newly-plastered walls. The vegetable and many of the artificial pigments are the worst offenders, leaving the selection between the relatively few mineral pigments. A committee appointed by the National Association of Cement Users (U.S.A.) has reported that the only mineral pigments that should be used are comprised in the following list: Lampblack, manganese dioxide, red iron oxide, English red oxide, brown roasted iron oxide, brown ochre, yellow ochre, ultramarine, chromium oxide, ultramarine green and violet oxide of iron. This brief list of pigments does not allow of much latitude. The committee further reports that the amount that can be safely used is small owing to the danger of impairing the strength of the concrete. Five per cent. by weight

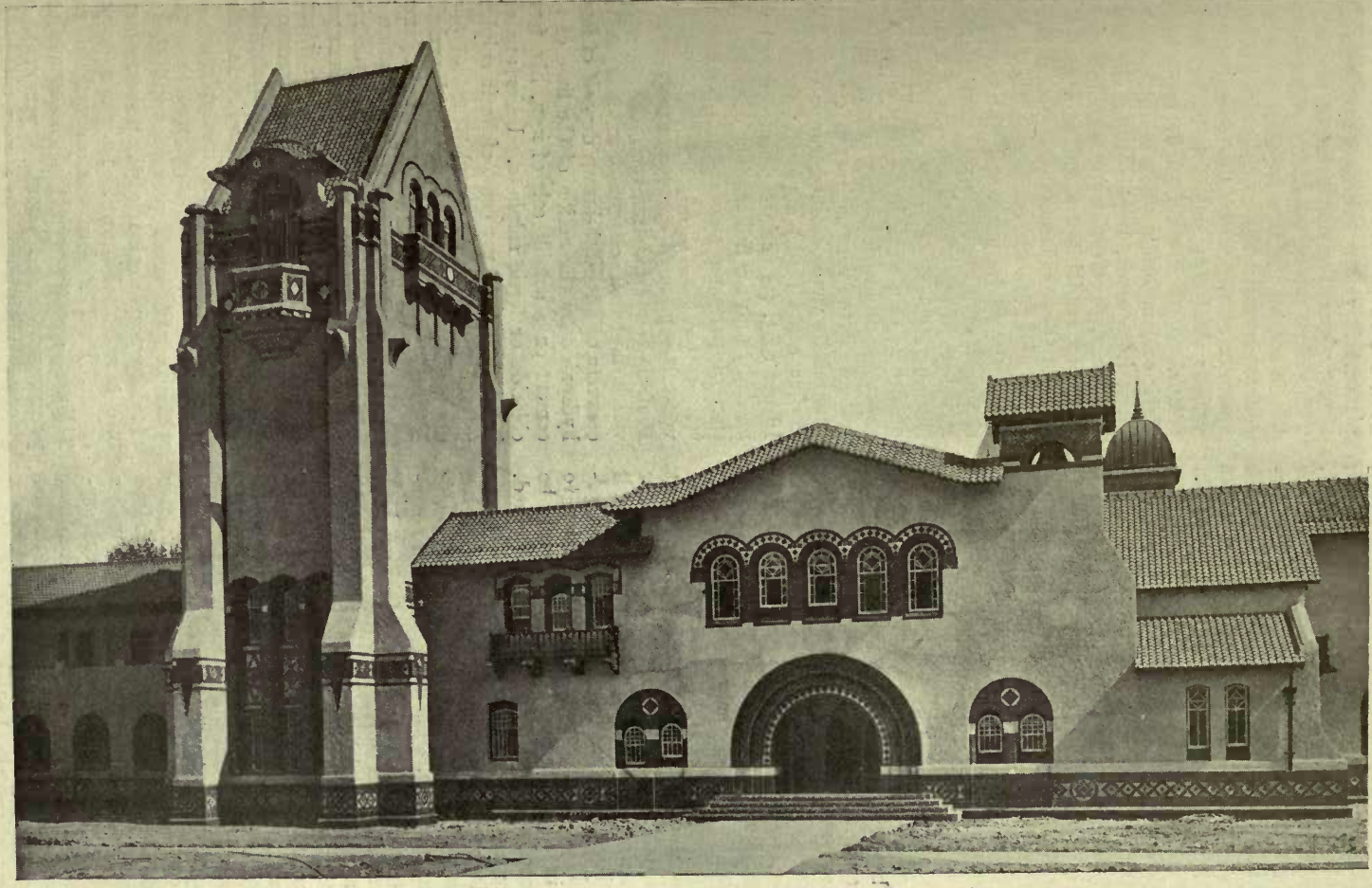


Fig. 469.—Californian State Normal School at San José

to that of the cement should be the limit, beyond which the impairment of strength is too great to justify a larger amount, except in special circumstances, as, for example, in a thin facing. Even this proportion of those pigments whose colour differs little from that of the cement does not produce a marked change. Lampblack, it is pointed out, is the principal pigment used for darkening, on account of its strong contrast and the fineness of its particles. For lightening the colour somewhat lime is the best material to use. Colouring matters can be used in either a dry form or as a paste. It will generally be found most convenient for mixing to use the dry form, thoroughly mixing it with dry mortar before the addition of any water.

The pigments exercise an influence on the setting properties. Crimson lake (alumina base) and barium chromate quicken the cement; manganese dioxide, red ochre, and chinese red retard it; and ferric oxide, yellow ochre, ultramarine, and chromium oxide have a slight quickening effect.

In colouring concrete by the admixture of pigment, the resultant tint must not be judged by that of the wet mixture, the concrete always drying out lighter than that would suggest. The best course is to mix together small measured quantities of all the ingredients dry, add the water, thoroughly incorporate, and allow a portion to set in a mould; the experiment can be repeated until the desired result is obtained.

For a white concrete, use 1 of white portland cement and 2 of hard marble, screening to pass through a No. 8 screen and be collected on a No. 40 screen. When the concrete is hard, in three or four days, it may be rubbed with a terrazzo stone to polish the marble particles.

Before giving a table of concrete colouring pigments, it is desirable to summarise the experiments carried out in the United States by Prof. Charles E. Pellew with the object of finding the most suitable pigments for colouring concrete. The question of expense was of vital importance, and in the matter of first cost it was evident that surface colouring would be cheaper. But, unless there is a thoroughly hard permanent surface for the pigments to adhere to, and to obviate as far as possible the use of linseed oil, upon which the lime exerts a strong action, the body colouring is probably the most satisfactory. For *yellow*, Prof. Pellew thinks that the only available pigment is yellow ochre, 8 per cent. of a strong pigment giving a bright tan colour. This yellow can be used for shading the red, but is not so effective for this as the manganese brown. Yellow ochre with small amounts of permanganate brown will give various shades of yellowish brown or buff colour. *Black* is obtained by using carbon black or lampblack. For a bluish shade a black iron oxide, imported for the use of gas works, gave satisfaction. The only *red* pigments available are red oxides of iron, some of them natural, finely ground hæmatites, and others artificial. They differ greatly in shade, price, and staining power, the cheaper pigments being unsatisfactory. It is best to use a small amount of a strong though high-priced pigment than larger quantities of a weak but cheap one. After experimenting with twenty or more different colours from various manufacturers, the best results were obtained from a red colour at five cents (2½d.) a pound; from 7½ to 10 per cent. (of the weight of cement) was needed to give a full shade. A slight addition of permanganate brown gave a red terra-cotta shade. As regards *brown*, a vegetable pigment proved unsatisfactory. Prof. Pellew tried iron-rust, the action of which is based on the formation in the concrete of a reddish-brown deposit of ferric hydroxide by the action of the lime of the cement on a soluble salt of iron, like ferric chloride or ferric sulphate. Unfortunately, it takes 25 to 30 per cent. of ferric sulphate (of the weight of concrete) to get at all a decided colour with this compound, and this is a serious drawback. He tried manganese brown, which is based on the formation in the concrete of brown manganese hydroxide by the reduction of the salt potassium permanganate. The latter possesses a strong rich purple colour, which in the presence of oxidisable material, such as organic matter, turns at once to a full seal brown. In staining concrete the organic matter must be supplied in the form of glucose or sugar, which in quite small quantities will change the deep purple colour of the permanganate into a rich seal brown. To get a deeper colour, Prof. Pellew was obliged to use 24½ per cent. (by weight of the cement) of permanganate and about ½ per cent. of glucose. With regard to a *green* colour the high price of chromium oxide, the only green mineral pigment that will stand the action

of lime, prevents its general use. In the absence of a strong blue that will stand the action of lime, it is not possible to obtain a good green by modifying the colour produced by yellow ochre. Ultramarine blue is fast to lime, but its staining power is low when mixed with other pigments, while Prussian blue is easily attacked by the cement, and is not, therefore, included in the table of pigments given on this page. It seems probable that for greens some form of surface colouring will have to answer, but much remains to be done in the way of experiment.

TABLE OF PIGMENTS

The weights of pigments given in the following table are those recommended for adding to each cubic foot, reckoned as 90 lb., of cement; but the table must not be taken quite literally, as experiments have obtained widely different results. No greater claim can be made for the table than that it will act as a rough guide. It cannot do more, owing to the great differences frequently existing between two pigments passing under the same name. They may be unlike one another chemically, physically and in their percentage content of inert cheapener which so seriously affects the staining power. The pigments mentioned will certainly give the tints, but the proportions must be adapted as required, depending, as above suggested, on the strength of the pigments themselves and also on the proportions of the concrete.

Black	10 lb. of manganese dioxide or lampblack.
——, Bluish	8 lb. or 9 lb. of black iron oxide.
Blue	3½ lb. to 4½ lb. of ultramarine.
——, Violet	4½ lb. of violet oxide of iron.
Brown	4 lb. of brown ochre.
——, Chocolate	6 lb. of manganese dioxide, 4 lb. of red oxide of iron, and 2 lb. of black oxide of iron or copper; or 3½ lb. of burnt umber.
Buff	3½ lb. of yellow ochre; or 4½ lb. of yellow ochre and 2¼ lb. of permanganate brown.
Green	10 lb. of chromium oxide or ultramarine green.
Grey	½ lb. to 3½ lb. of black or manganese dioxide.
Pink	3 lb. of crimson lake (alumina base).
——, Dull	2 lb. to 3½ lb. of venetian red.
Pinkish Brown	½ lb. of burnt umber.
Red, Brick	2 lb. to 3½ lb. of Chattanooga iron ore or red iron ore.
——, Bright	5 lb. of Pompeiian or English red.
——, Dull	5 lb. raw iron dioxide; carbon black may be added.
Slate	½ lb. to 3½ lb. of lampblack or manganese dioxide.
——, Blue	1 lb. to 3½ lb. of ultramarine blue.
——, Pinkish	½ lb. to 1 lb. of Venetian red; or ½ lb. of Chattanooga iron ore or red iron ore.
Tan	7 lb. of yellow ochre.
Terra-cotta	English red with permanganate brown; or 2 lb. to 3½ lb. of Chattanooga iron ore or red iron ore.
Yellow	5 lb. to 10 lb. of yellow ochre.
White	44⅓ lb. of barytes (barium sulphate).

Stains.—Manufacturers have placed upon the market a number of stains suitable for use on concrete. A brown stain is easily made by dissolving iron sulphate (green copperas) in water, the proportion being about 2½ lb. per gal. The colour is strengthened by repeating the application. The addition of alum to the above solution gives a pale yellow; whilst chrome alum gives a green. Stains have the advantage over paints that they do not cover the concrete with a thick coating, but sink into the pores and preserve the natural texture, while at the same time colouring it in rich, deep tones. They can be applied easily and rapidly, their covering power is two or three times that of oil paint, and they cost, bulk for bulk, much less.

Distempering.—Concrete can be successfully distempered with common whitewash or with cement washes; as regards

colours, the pigments used must be fast to lime (see under the heading "Body Colours"). It will be better to kill the alkali in the concrete surface with dilute acid and then well wash with water before applying the distemper.

Painting.—By this term, of course, "oil painting" is understood. Painting is desirable only on smooth concrete, the constructional nature of which it is not desired to emphasise. The paint forms a film of metallic oxides, sulphates, etc., and oxidised linseed oil on the face of the work, closing the pores and undoubtedly adding to the weather-resisting properties. Unfortunately, the lime exerts a violent chemical action upon linseed oil, and special preparation of the concrete surface is absolutely essential. Usually, this preparation consists in killing the alkali and then coating with at least two coats of good white-lead paint before applying the finishing coat. But a different style of treatment was suggested some time ago before the American Society for Testing Materials by Charles Macnichol, who spoke from the experience of many years. He primes the concrete with zinc sulphate ground up with an equal quantity of water. Then, after two or three days, the concrete can be painted on safely. Dr. A. S. Cushman suggests the explanation that a chemical reaction results in the formation of gypsum (calcium sulphate) and zinc hydroxide (hydrated oxide of zinc), which substances, held within the pores of the cement, do not affect linseed oil injuriously—indeed, they are common paint pigments. The method is economical as regards the after coats, because the suction of the cement has been already satisfied.

The architect will usually hesitate to specify any particular formula to be followed in painting concrete, and will generally content himself by specifying one of the several compositions on the market that are specially made for the purpose, the constituents of which are, for obvious reasons, kept secret. Some of them are the result of very careful trials, and Dr. Macmimilian Toch, in a lecture delivered some time ago before the Paint and Varnish Society, hinted that the best results might be obtained by a proper admixture of Menhaden fish oil and tung or Chinese wood oil. Whatever paint or preparation is used, it is generally recognised that it is useless to apply it to green or fresh concrete, but an exposure of at least a year, or even two years, will render the surface fit for receiving the paint. In actual practice, however, it is usually desired to finish the building almost immediately after it is erected, and in such a case it becomes necessary to prepare the surface in order that the paint may permanently adhere. Even when the concrete has been exposed for a year or more the application of such a preparation may be made as a matter of precaution to neutralise any free lime which might remain.

Opinion widely differs as to the composition of this preparation. Many practical men advocate the use of a solution of zinc sulphate, commonly known as "white vitriol." This is used in the proportion of 8 lb. to 1 gallon of water. It is urged against this method that something more than a neutralisation of the free lime is required, and that to obtain satisfactory results a material should be used which will fill up the pores of the concrete and prevent suction. Hence there are advocates of compositions in which paraffin wax plays an important part, a method, it may be remarked in passing, which was successfully employed a year or so back in connection with Cleopatra's Needle on the Thames Embankment.

Under ordinary conditions, however, the most practical method is to apply a liquid to neutralise the free lime, and, that object being thoroughly effected, the paint might be ordinary white-lead, although much better results would be obtained by the use of one of the special "concrete paints" already referred to.

Among the solutions that have been recommended for use under the coating of paint are the following: For concrete floors the use of sodium silicate has been suggested. Dr. M. Bennett Blackler advocates the use of copals made up with a small quantity of oil and diluted with turpentine, and he expresses his opinion, also, that casein solutions treated when dry with formaldehyde would give a coat which is absolutely impermeable to water and form a surface on which paint would stand. In these opinions Dr. Blackler has been backed up by many other eminent men; in fact, it is not improbable that casein forms the active base of many of the specialities now on the market.

Assuming that the surface is in as good a condition to receive the paint as can be

expected, the next question arises as to what is the best paint to apply. Many people believe that when the walls are in a proper condition the ordinary paint can be used without difficulty, and the following formula has been published:—For the priming coat: 100 lb. of pure white-lead, 9 gals. of pure boiled linseed oil, and 1 gal. of turpentine. This may be followed by using instead of the boiled oil 9 gals. of raw linseed, and 1½ pints of turpentine drier. Another priming coat recommended consists of 85 lb. of pure dry red-lead mixed with 1 gal. of boiled linseed oil, and ¼ gal. of turpentine. The body coat should in this case be 100 lb. of pure white-lead, 4 gals. of pure linseed oil (one third boiled and two-thirds raw), and the finishing coat 100 lb. pure white-lead, 3½ gals. of pure linseed oil (one-third boiled, and two-thirds raw), and 1 pint of pure turpentine. The red-lead in the priming coat above mentioned would probably be in most cases objectionable because of its colour.

When the surface of concrete is covered with cement, the latter may be painted by giving at first two coats of Alabastine, proceeding with oil paint in the ordinary way. W. G. Scott recommends as a paint for cement the following: 10 lb. plaster-of-paris, 2 lb. portland cement, 8 lb. whiting, 4 lb. wheat flour, 6 lb. zinc oxide, and 2 lb. red-lead. This mixture should be ground in 1 gal. of prepared wood oil, and ¼ gal. of soya bean oil. The pigments are mixed separately with the oil, adding them in the order given, and if the paste is then found to be too stiff a little more of the bean oil is added. This paste, Mr. Scott says, grinds with some difficulty, and it is better to mull or run through an iron mill first, and then to grind fine in a stone mill. In use, the paint is thinned with benzine or turpentine, or with a mixture of 7 parts of turpentine and 1 of paraffin oil. The prepared wood oil referred to may be made by stirring 9 lb. litharge and 1 lb. of manganese borate into 50 gals. of Chinese wood oil, and heating for five hours at a temperature of 350° F. (177° C.). After the litharge and manganese borate have settled out and the oil is clear, it is ready for use. This paint or cement is intended for application to porous cement, and may be used on concrete. The same author gives the composition of a marine paint which will set under water, and which may be quoted here: 10 lb. portland cement, 5 lb. silicate, 3½ lb. zinc oxide, 7½ lb. red-lead, 5 lb. litharge, and 2 lb. graphite. These are mixed with 1 gal. of boiled linseed oil and $\frac{1}{16}$ gal. of paraffin oil. It sets under water, and becomes in time as hard as stone. The same mixture if thickened with whiting or plaster-of-paris forms a valuable cement for many purposes.

Some careful experiments conducted by Mr. Henry Gardner, assistant director of the Institute of Industrial Research, Washington, D.C., are reported in his "Paint Technology and Tests." He carried out his tests on mortar made of 1 part of portland cement, and 3 parts of sharp clean sand. Omitting the failures, the results were as follow:—No. 1. Concrete primed with a 25 per cent. solution of zinc sulphate crystals dissolved in water. A wide brush was used for the application, and the spreading rate was approximately 200 square feet per gallon. Second and third coated on the second day with the following composition: Sublimed white-lead (similar to "Purex" in Great Britain) 50 per cent., zinc oxide 35 per cent., silica and barytes 12 per cent. and Prussian blue 3 per cent., ground in linseed oil, turpentine and drier. This panel after three years' exposure was in good condition, although there was a slight checking. In test No. 2 the concrete was treated with a 20 per cent. solution of alum (aluminium sulphate), and a paint of the same composition as before. The results were almost identical with No. 1. In other tests the concrete was primed with a solution made by dissolving 10 parts of sodium oxalate in 100 parts of water. The second and third coats were linseed oil paints in red, brown, blue and green. The results are reported as being "very good." But even better results were obtained when the second and third coats were composed of zinc oxide and barytes ground in an oil (kind not given) having "a low saponification value." This paint dried very slowly, but the results are reported as "excellent, extremely glossy waterproof surface presented."

Durability of Reinforced Concrete

When, recently, a reinforced concrete house, built fifty-eight years before at St. Denis, France, by François Coignet, was examined and the work cut away to disclose the internal condition, both steel and concrete were found to be in an excellent state of preservation. It is a practical illustration of this kind, vouched for by a deputation from the (British) Concrete Institute, that carries conviction to the minds of those engineers who are unmoved by mere theories and by hundreds of laboratory experiments. That concrete has an effective life of hundreds of years under favourable conditions is an established fact; further, those who have made a special study of reinforced concrete know that in ordinary circumstances the steel will last as long as the concrete in which it is embedded. The concrete exercises a protective and preservative action on the steel; and while he concrete gains in strength a hundredfold, the steel gains in durability a thousandfold.

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Blows and Shocks.—Concrete is not an ideal material to employ in situations where heavy direct blows are likely to fall upon it. Obviously, its brittleness is its greatest defect, and it therefore follows that while it makes a durable and wellnigh everlasting floor for ordinary traffic, it is not suitable for the floors of workshops in which heavy iron implements are carelessly dropped about, or in which heavy trucks with flat-rim wheels are used, as rough treatment soon chips the surface. Truck wheels need to have curved treads or rubber tyres, so that there are no edges to dig into the concrete when turning a corner, even when a truck is tilted considerably. Hence, although concrete makes an admirable floor foundation, it is frequently desirable to have the actual floor of wood, or to let in iron plates flush with the surface where strongly abrasive wear is likely to occur. As a floor material, too, concrete suffers with all stones, whether natural or artificial, in lacking in resilience, the "deadness" of the floor causing the people who stand or walk on it to become tired sooner than would be the case on a wooden floor.

Reinforced concrete is being increasingly used for dock and harbour works, in which situations it is commonly subjected to great shocks. Mr. F. E. Wentworth-Shields (of the New Dock Works, Southampton) gives a valuable opinion with regard to its suitability for such applications. "If reinforced concrete is to be used for marine work," he says, "it must be carefully fendered at all parts where it is likely to be struck. In most systems of reinforced concrete, the steel is placed within an inch or two of the surface; and if the structure is not protected by timber fenders, this concrete skin is easily knocked away, leaving the steel naked and liable to rust. At the same time it is true that steel-concrete does not spall easily from blows, provided they do not fall directly on the concrete, and it will stand a wonderful amount of shocks, and bending due to shocks, if a wooden fender is interposed. The coaling-jetty at Southampton probably suffers as much from the blows of ships and barges as any steel-concrete structure yet made. It is protected by fenders of American elm, and in spite of blows which have caused the whole jetty to sway, there is no sign as yet (two and a half years after completion) of the concrete spalling off, except at one or two places where the blow has fallen directly on the concrete." The same writer gives an interesting example of the ease with which reinforced concrete coast constructions can be repaired—the ease with which repairs can be effected being important when broadly viewing the subject of durability. He states that a steamer collided violently with the quay of this same jetty, breaking two piles and the beams which they supported. "On examination by divers it was found that the piles were broken down to a level of 12 ft. below low water. The work of repairing the quay, however, turned out to be fairly simple; the broken concrete

and steel rods were cut away by divers, who also erected new rods to take their place. These new rods were joined to the undamaged part of the old ones by steel tubes, and the stirrups or binding wire were then placed round the new rods at correct intervals. A water-tight casing of timber was then placed round the pile, which extended from the undamaged portion to above low water. From this casing the water was pumped out and the concrete was filled and rammed into the box in the ordinary way. Above low water the piles and beams were remoulded in just the same way as they had been originally built."

Earthquakes.—These subject buildings to every form of stress and shock. Parts of a building designed to take simple compression are, in a moment, subjected to enormous bending strains, and floors are suddenly called upon to act as struts. Whilst no form of building can long resist severe earthquake attacks, reinforced concrete has given as good an account of itself as any other material, and a better account than many. There is, unfortunately, a divergence of opinion on the subject, and, as might naturally be expected, the brick and terra cotta interests have made out a strong case for the superiority of their own materials. Captain Sewell, who personally studied on the spot the results of the San Francisco earthquake shortly after the catastrophe, stated in an article in *Concrete* that at "Palo Alto, at the Leland Stanford University, there was a museum building which consisted of three wings: the central one of reinforced concrete, and the two side wings of brickwork, with reinforced concrete floor systems. This building was not far from the line of the fault which caused the earthquake, and it received a severe shaking. Externally, the reinforced concrete wing appeared to be absolutely uninjured, except that some statues were shaken down from the front parapet wall. The two brick wings were almost in a state of collapse. An examination of the interior showed that some damage had been done in the reinforced concrete building, in the shape of a few cracks here and there; but the sum total of structural damage was insignificant, and the writer is inclined to think that one thousand dollars would cover all the damage to this wing. The damage to the brick wings is at least from 50 per cent. to 75 per cent."

In 1908 there was a terrible earthquake disaster at Messina, in Southern Italy. G. Flament Hennebique, in a paper presented to the French Society of Civil Engineers, described how certain Hennebique structures at Messina had withstood the shocks. The more noteworthy instances are here mentioned. A roadway over the Portalegni stream remained intact in spite of the débris which accumulated upon it; the floors of the Mandalari Hospital contributed largely to preventing the collapse of the building; a 4,000 cub.-metre reservoir remained perfect; the walls of the Messina museum collapsed and carried with them the floors, which remained unbroken; the church of the Madeleine collapsed, but the reinforced floors of the adjoining house remained intact; the floor of the railway electric generating station, situated in the most severely shaken part of the town, did not suffer, although the upper parts of the building collapsed or were badly cracked; the hospital attached to the medical school was destroyed, with the exception of the reinforced concrete parts, notably the staircase, which remained standing in the ruins.

Settlements.—In a milder, slower, but actually surer way, settlements subject a building to the same kind of destructive stresses as those caused by earthquake. An appreciable settlement will inevitably lead to cracks, unless it is an even sinking over the whole area of the site. In a wooden or steel structure there is a slight accommodation to the consequences of a local settlement, on account of the elasticity of the material; but there can be none in a monolithic building. The fact to be borne in mind is that the foundation should be of such a nature as to put the risk of settlement out of all question; and the use of properly driven piles or of a reinforced concrete raft easily ensures that condition of affairs.

Frost.—This is a deteriorating influence which is at work upon most building materials. Few substances containing calcareous compounds are free from its destructive effects. Extremely porous substances are more liable than close-grained ones to be destroyed by a disintegration of their particles due to the freezing and consequent expansion of absorbed water. In the case of concrete, however, the effect of frost is only serious under certain limited conditions. The greatest danger likely to occur from frost is while the concrete is being gauged up and

before it has set, and, when necessary, special precautions must be taken at this stage. The effect of frost after the concrete has set is of little consequence, as reinforced concrete of good quality absorbs relatively little moisture. Where defective cement has been used, frost might constitute a very real danger. Valuable information with regard to the effects of frost upon concrete has been obtained by submitting briquettes of the material under varying conditions to different degrees of cold, and noting the tensile strength after certain periods have elapsed. Should a frost occur immediately after mixing, the strength of the concrete may be permanently affected; but should it occur after twenty-four hours and before the end of the eighth day, and then be only slight, the setting will be retarded, but normal strength will be developed later. After the eighth day, the frost has no effect, but heavy frost of, say, 15° is likely to injure freshly mixed concrete permanently. It is wise to refrain from mixing cement or concrete when the temperature is below 29° F., which is the freezing point of the material. The detrimental effects of frost appear to be due not to chemical difference or changes, but purely to mechanical or physical ones, as before the cement has had time to exert its binding action upon the aggregate, the frost has already robbed it of its power to undergo the chemical changes characteristic of "setting."

Much of the effect of frost depends on the amount of water present in the concrete, a very wet specimen not being nearly so seriously injured as a dry one. No satisfactory explanation of the action of frost has yet been made; it appears to be due to the great thrust exerted when the water crystallises from the liquid state, but the nature of this "crystallising force" is, as yet, but imperfectly known. It appears to be much more powerful than the increase in volume which water undergoes on freezing and to which the disruptive action of frost is commonly attributed.

Fire.—The ability to resist the destructive effects of fire is one of the most important factors in determining the utility of a building material. The cement in concrete is ruined when its water of crystallisation has been driven off, this occurring at temperatures between 600° and 800° F. (316° and 427° C.); but if the dehydrating action is slow, the material that has undergone it generally remains and insulates the rest of the work from the heat. The risk, however, of extensive damage from this cause is not great, since serious dehydration requires an amount of time which would not be likely to have elapsed before the action of the heat would be quenched by water applied by the firemen.

It is impossible to attempt even to outline here the numerous experiments conducted under eminent authorities to test the fire-resisting properties of concrete; little more can be done than to say that the result of the many accurate experiments has been such as to justify the conclusion that reinforced concrete is one of the very best fire-resisting building materials. Tests conducted by the British Fire Prevention Committee have shown that reinforced concrete suffered no diminution of strength or decomposition to any appreciable extent by the heat, although the temperature was raised to as high a point as 2,000° F. (1,093° C.). In 1906 Prof. Woolson subjected cubes of concrete to a temperature of 1,500° F. (816° C.) for two to five hours, and he found that, with the exception of specimens in which quartz gravel had been used in place of broken stone, the interior of the concrete did not attain the exterior temperature—convincing proof of the low thermal conductivity of concrete, and pointing to the fact that when a concrete wall is heated on one side no serious rise of temperature is occasioned on the other side.

Concrete blocks two months old were placed in boiler flues for twenty-one days and subjected to temperatures of 1,250° and 250° F. (677° and 121° C.) night and day alternately; there was practically no alteration in the condition of the concrete at the end of the period.

During a fire that occurred recently in a large warehouse in Dresden, in spite of the fierceness of the flames, the firemen were able to keep the fire within the story in which it had originated, owing to the reinforced concrete construction. Subsequent examination showed that the floor was practically unaffected, although it had been exposed to a temperature of nearly 2,010° F. (nearly 1,100° C.), and only in one or two places had small parts of the concrete fallen off, laying bare the reinforcement, in spite of the fact that it was acted upon while at these high temperatures by the firemen's hose.

A few years ago the United States

Geological Survey tested thirty panels of various building materials to determine the effects of fire and subsequent quenching with water. In most cases the materials were subjected to the direct application of gas furnace heat for two hours and then quenched with water. Figs. 470 to 475 are photographs of six of the panels after quenching. The explanations under the figures should be read before attempting any comparisons between them. The report (by Richard L. Humphrey, an officer of the Survey) contains a number of significant statements, of which the more important are here quoted:

"Much of the damage done to the building materials in this series of tests was occasioned by internal stresses, the gas flame heating one face of the test pieces much more rapidly than the other face. All the materials tested, including hydraulically pressed brick, cracked more or less. *The concrete cracked least*, but the tests indicate the necessity for using metal reinforcement in concrete walls to distribute the effect of the expansion.

"The average temperature attained by the faces of the panels ten minutes after the gas was lighted was about 324° C. (615·2° F.), and nearly half of the panels had been subjected to freezing weather just prior to the tests. The average temperature of the face of one panel of building blocks rose from 0° to 450° C. (32° to 842° F.) in the first ten minutes of firing, while that of another panel of the same material ranged from 22° to 600° C. (71·6° to 1,112° F.) during the same interval.

"A fact brought out most clearly by these tests is the low rate of heat-transmission in portland cement mortars and concretes. This is one of the desirable qualities in materials intended for fireproofing purposes. For marking the cement blocks, linen tags were fastened by wire nails to the interior walls at the time of moulding. Most of these tags remained in place during the test, and when the walls were dismantled the tags in every case were found entirely undamaged.

"There was a comparatively slight increase in the temperatures of the backs of the blocks during the test. The rise in temperature of the backs of the building blocks made of river-sand mortar varied from 25° to 40° C. (77° to 104° F.), while the rise in the average temperatures of the faces of the cement blocks ranged from 650° to 900° C. (1,202° to 1,652° F.). The backs of the mortar blocks made of slag sand showed a rise of temperature of only 10° C. (50° F.), while the faces were heated up to 800° C. (1,472° F.). The low rate of heat conductivity of the mortar blocks is shown by the slowness with which the temperature of the backs of the panels increased in comparison with that of the faces.

"The damage done to the faces of the mortar and concrete panels would probably be caused at a temperature about half that of the maximum temperatures reached, owing to the water of crystallisation being driven from a layer of the mortar.

"The backs of the brick panels also showed a small increase in temperature, but the natural building stones and the tiles proved poorer non-conductors of heat. The temperature of the back of a panel composed of plastered tiles increased to 128° C. (262·4° F.).

"It was difficult to determine whether the limestone, granite, gravel, or cinder concrete sustained the least damage. The faces of all the panels were more or less pitted by the fire and washed away by the stream of water. The test was unfair to the cinder concrete, as the cinder was very poor, containing a large percentage of unburned coal; however, the sample selected was the best of the six or eight investigated when purchasing. During the fire test the coal ignited and left the surface of the concrete very rough and badly pitted. The limestone aggregate in the face calcined, and the granite aggregate split and broke away from the surface mortar. The granite concrete probably behaved the best. The damage in no case extended deeply, probably not more than 1½ in. The evidence shows that even at this depth the temperature was comparatively low. The high stresses produced in the panels by the rapid rise of temperature of the faces while the backs remained cool caused cracks. On taking down the panels the blocks of concrete were found to be cracked vertically for some distance back from the face.

"The serious damage to the various natural building stones precludes any comparison among them.

"The concretes tested were pieces of broken beams that had been tested in the Government's structural materials testing laboratories. The pieces were 8 in. by 11 in. in cross section and of various lengths. They were fired with the 11-in. face exposed. In making the concretes, four distinct types of aggregate had been used, limestone, granite,

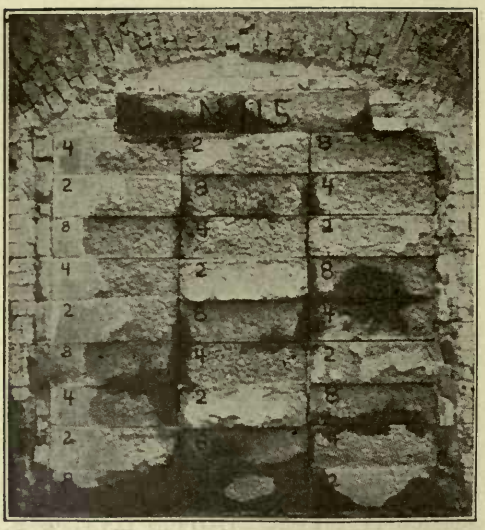


Fig. 470.—Single-air-space Mortar Blocks (Sand and Portland Cement) Laid in Fireclay, after Firing and Quenching

These 63-day-old blocks were fired for 121 mins. (max. temp. 700°—800° F.) and quenched for 5 mins. with water at 51° F. A strip to the left of the panel was not quenched. After cooling all the blocks were found cracked across the web. Surfaces of the blocks could be broken into small pieces by gentle tapping.

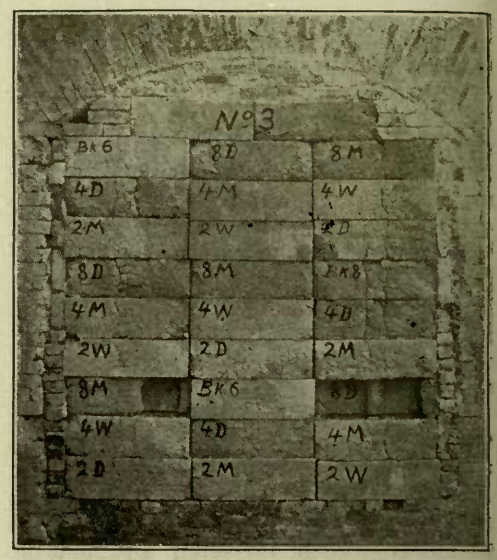


Fig. 471.—Double-air-space Mortar Blocks, after Firing and Quenching

Fired for 120 mins. (max. temp. about 650° F.), and, after lapse of 19 mins., quenched for 5 mins. After firing, vertical cracks showed on each side of the centre webs. Richer wet blocks withstood better than the lean damp blocks. In some cases, end webs split away from and parallel to faces of blocks, whereas the middle web remained intact and the face split away from it at each side.

Fig. 472.—Terra-cotta Three-air-space Partition Tiles Laid in Cement Mortar, after Firing and Quenching

Plastered with ordinary lime plaster ½ in. thick, and finished with a thin skim of plaster-of-paris. Fired for 121 mins. (max. temp. about 900° F.) and quenched for 5 mins. with water. Soon after starting firing, the plaster came away, and cracks developed. In removing the panel from the furnace, the door stuck half-way, and the panel was badly shaken in endeavouring to prise the door open. Thirty-seven mins. elapsed before the water was applied, the water washing away about 65 per cent. of the tiles that were exposed directly to the stream.

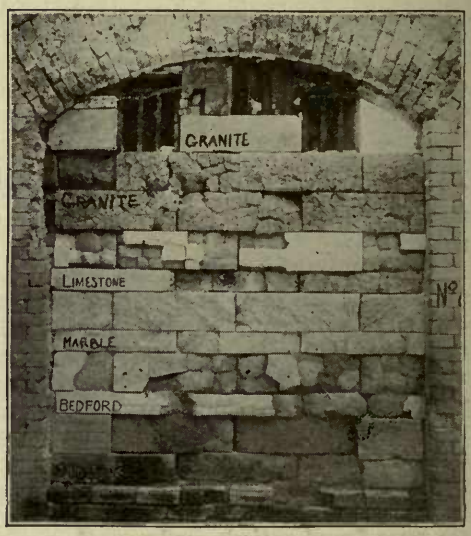


Fig. 473.—Various Building Stones Laid in Cement Mortar, after Firing and Quenching

"Bedford" indicated in the illustration refers to Bedford limestone. Fired for 120 mins. (max. temp. about 850° F.) and quenched for 5 mins. with water at 57° F. During firing, the sandstone spalled; the limestone spalled and developed fine irregular cracks; nearly all the granite blocks showed small vertical cracks; the Bedford limestone showed small vertical cracks about 8 in. apart; the marble showed signs of calcination; and the wall bulged 1 3/16 in., this being reduced after quenching to ⅞ in. After cooling, the granite, limestone and Bedford stone were cracked and had the faces badly washed off.

gravel, and cinder. The aggregate was sized to pass a 1-in. screen and be retained on a ¼-in. screen. The concretes had been mixed in proportions of 1 of cement, 2 of Merrimac river sand, and 4 of aggregate, and were of medium consistency."

F. E. Wentworth-Shields, in the paper already quoted from, says it appears "that for fire-resisting purposes it is best that the steel reinforcement should consist of many bars of small section in preference to few bars of large section, as the first arrangement forms a close network which prevents pieces of concrete from dropping out and so reducing the strength of the building. It is also important that the concrete aggregate be crushed small. The ideal composition for the aggregate seems to be a strong flint or granite for the hearting, with an outside skin of cinder concrete to act as a good non-conductor. It is essential, too, that in a building the walls, floors, and posts should be well tied into one another. There is a tendency, nowadays, in designing steel-concrete buildings to reject that material for the walls on the score of expense, using it only for floors and columns. Sometimes a compromise is made; for instance, at a tobacco warehouse built at Bristol on the Coignet system, the walls consist of a framework of steel-concrete with brick panels. The brickwork can be arranged, as at Bristol, to conceal the concrete entirely; but the collapse of any part of the brickwork would not endanger the floors or the concrete framework."

It will be noted that cinder concrete is recommended for the outer face—a recommendation which the results of the United States Geological Survey tests do not seem to endorse. In the earlier chapter on concrete a note is made of the difference of opinion regarding this material; but not only do opinions differ, even experiments appear to give contradictory results. For instance, fire tests of cinder concrete walls were conducted at Columbia University over a period of at least two years. The walls of the buildings used in subjecting floor constructions to fire tests were given five fire tests of four hours each, during which the temperature was raised to about 1,700° F. (nearly 930° C.). At the end of each test a stream of water at 60 lb. nozzle pressure was played to and fro over the ceiling for ten minutes, while the ceiling wall was red hot.

Fig. 474.—Concrete Blocks Laid in Fireclay, after Firing and Quenching

Concrete was of four kinds—namely, limestone, cinder, granite, and gravel, all being mixed with portland cement and river sand. Fired for 123 mins. (max. temp. slightly exceeding 800° F.) and quenched for 5 mins. with water at 52° F. During firing small pieces fell from the faces, and the limestone, gravel, and cinder concretes were pitted, the latter two suffering the most.

Fig. 475.—Granite Blocks Laid in Cement Mortar, after Firing and Quenching

Firing for 120½ mins. (max. temp. 900° F.) and quenched for 5 mins. with water at 57° F. During firing, panel bulged 3 in. and some of the joints and blocks cracked. Three blocks showed signs of spalling. During quenching, spalling and cracking became general. After cooling, the surfaces of the stones that remained intact could be easily rubbed off with the fingers: a gentle tapping broke the blocks into fragments.

After this severe treatment, the walls, according to Prof. Ira H. Woolson, were practically as good as they were when new, except that the rear wall was scored by the water to a depth of from ½ in. to 1 in. for 2 ft. or 3 ft. near the top. Prof. Woolson said he believed the structure to be good, with occasional plastering, for an indefinite number of tests. In his opinion, the percentage of coal and the amount of fire material in cinder concrete have, within reasonable limits, very little if any effect on its fire-resisting qualities. He believes that sifting and washing the cinders would be a useless expense, and injurious to them as an aggregate for concrete. The pieces of coal which were close to the surface in the walls were burned to an ash, which remained in place and acted as a non-conductor of heat. Plenty of particles of pure coal could be found in the walls less than 2 in. beneath the surface.

It is well known that the water used in extinguishing a small fire generally does more damage than the fire itself. Where there is much wood and plaster construction, the water swells the wood and brings down the suddenly quenched plaster; it pours through the ceilings and does infinite harm in the floors below; on the subsequent shrinkage of the wood there is further trouble, and more plaster is cracked. On the other hand, the quenching of a localised fire does not appreciably affect the reinforced concrete, and as the floor is watertight (practically considered) there is no harm done to the room below.

CHEMICAL DESTRUCTIVE INFLUENCES

In determining the life of building materials and their powers to withstand the many influences which lead to decay, the test of time is by far the most accurate and satisfactory; but the careful study of the chemical constituents of a building material may sometimes lead to a means of minimising the effects of destructive influences. It will be shown in this section that while waters or atmospheres impregnated with chlorides or sulphates and acid-reacting substances have generally but little effect upon concrete, there are many cases where a special knowledge of such actions may be desirable, and, in fact, essential, as in the construction of chemical factories, public conveniences, stables, etc., where reacting gases are likely to permeate the material of the building. In such circumstances, ample knowledge of the chemical action of certain gases and liquids would be useful in arriving at a means of preventing decay.

Atmosphere.—An average sample of atmospheric air has the following composition per one hundred volumes: Oxygen, 20·61; nitrogen, 77·95; carbonic acid gas, 0·04; aqueous vapour, 1·40; nitric acid and ammonia, traces; carburetted hydrogen and sulphuretted hydrogen, traces in towns.

The oxygen and nitrogen are without action upon concrete, but the former, in the presence of moisture, oxidises iron and steel, so forming "rust."

Nitrogen is one of the most inert substances known, and may be entirely disregarded so far as concrete is concerned.

Carbonic acid may slightly influence the condition of concrete. It is produced by the solution of a gas, carbon dioxide (CO_2), in water. The cement, which acts as a matrix in the gauging of the concrete, is composed chiefly of calcium alumino-silicates. There is also a certain amount of carbonate of lime (strictly, calcium carbonate), which is slightly soluble in water charged with carbon dioxide (3 parts by weight in 1,000), the solution containing the acid calcium carbonate, $H_2Ca(CO_3)_2$. The proportion of carbon dioxide in the air varies considerably with the locality, and in the vicinity of large factories appreciable quantities of it are brought down in solution in rain. In some cases this would be a factor in accelerating the decay of concrete in course of time; but, generally speaking, under ordinary atmospheric conditions it may be entirely disregarded.

Nitric acid, of which the atmosphere contains traces, is a fuming, powerful liquid (HNO_3), which, when brought into contact with alkaline and basic substances causes chemical action to take place, resulting in the decomposition of the substances acted upon; but the proportion of this acid present in the atmosphere is too small for it to have any appreciable effect upon concrete.

The other substances mentioned as constituents of the atmosphere are in far too small proportions to be of consequence in destroying concrete; and, speaking generally, therefore, the atmosphere has no chemically destructive influence upon concrete, a statement substantiated by the stability of concrete after hundreds of years of exposure. Of course, where concrete is

used in buildings in which it may be exposed to any gaseous fumes, as in the manufacture of chemicals, the action of these gases upon the concrete requires special consideration.

Whether or not the moisture in the air affects the steel reinforcement will be considered under the next sub-heading.

Water.—Many authorities have feared that water must inevitably corrode the reinforcement and bring about failure, the danger being all the more serious because the steel is completely hidden from view, and the course of the corrosion cannot be watched. The fear seems a reasonable one until certain proved facts relating to the chemistry and physical conditions of concrete and steel have been stated. In the first case, a thin skin or shell of concrete is not waterproof when new, although it may become so in the course of a couple of years or so. Naturally, the better the concrete the less will be its porosity ; but even the best concrete will absorb a very small amount of water, which naturally may find its way to the steel. There are now two facts to remember :

(1) *Iron or steel cannot rust when damp except free oxygen be present.* The concrete used in reinforced work should be of high quality and therefore of minimum porosity. Although it may permit the passage of an extremely slight amount of water, air at atmospheric pressure could not penetrate it, and rust cannot therefore form. But perhaps it will be said that the microscopic particles of water may carry entrained air with them. While this is scarcely likely—probably quite impossible—it could still be granted, and absolute reliance placed on the next point.

(2) *Steel cannot rust when embedded in a strongly alkaline substance.* In the setting of portland cement a relatively large quantity of caustic lime (lime hydrate) is produced, and the concrete becomes saturated with it. This is strongly alkaline, and by decreasing the number of hydrogen ions present makes any corrosion of the steel impossible.

On the other hand, if the concrete were acid instead of alkaline, the number of hydrogen ions would be increased, and, assuming the presence of oxygen, the steel would be corroded. Iron rust, it will be remembered, is not simply an *oxide ;* it is a *hydrated* oxide, one formula being $2FeO_3 + 3H_2O$; this does not express the whole of the truth, since iron rust contains, besides the oxide, carbonic acid and ammonia. It is known that the composition of rust varies with surrounding conditions.

Of supreme importance is the necessity of using the best concrete obtainable. "Rotten," porous concrete, permitting the free passage of water, would not maintain the steel bright and free from corrosion, since the caustic lime is soluble in water, and in time would all be dissolved away.

Not only does concrete prevent the corrosion of the reinforcement, but it removes any rust that may have been present on the steel. For this reason no concern is felt over the use of rusty rods and bars, but that is a mistake. *Thick* rust reduces the sectional area of the steel and lessens the adhesion of the concrete, although it becomes completely absorbed.

It is claimed that experiments have proved that steel reinforcements to concrete are found to corrode rapidly when an electric current is passed through them. Such corrosion can take place only when the interior of the concrete is damp (that is, when the concrete is inferior), and need not be feared when material of the proper quality is employed. The rods, in the presence of moisture, would act as poles in a battery, and hydrogen would congregate round some and oxygen round others—a condition conducive to rust.

From certain authoritative experiments, it appears that the only risk of reinforcements rusting is likely to occur after the material has been stressed to 35,000 lb. per square inch, a condition of affairs which ordinary designing renders absolutely impossible in normal circumstances, since the tensile strength of structural steel itself is regarded as about 60,000 lb. per square inch ; the elastic limit is taken at about half that, and the factor of safety is 4 or 5. It is obvious that for rusting to be made possible such a stress must be applied as will give the steel a permanent set, and, by reducing its diameter, destroy—or, at any rate, seriously impair—the cohesion between the steel and concrete. Such a stress cannot occur unless the designing be absurdly and obviously at fault.

It will be of advantage in this place to give the conclusions arrived at by Ernest R. Matthews, A.M.Inst.C.E., after a number of tests, and presented by him in a paper entitled "The Corrosion of Steel Reinforcement in Concrete," read before the Society

of Engineers. Those conclusions are as follow: (1) Rusty steel embedded in concrete will in a very short time become bright, regardless of whether the concrete is in water or air. This point has, in Mr. Matthews's opinion, been conclusively proved by his experiments. (2) The application of cement grout to steel is an effectual safeguard against corrosion, but the greatest care should be taken in the grouting process to see that every portion of the steel is well coated; and before the steel is embedded in the concrete the cement grout should be allowed to dry on the steel. (3) If the aggregate used for the concrete is not porous and the concrete is well mixed, the reinforcement being well embedded, no cement coating is needed. This is proved by experiment. (Seeing that the application of a coat of cement grout is such an inexpensive procedure, Mr. Matthews makes it a rule in carrying out work of this kind to have all reinforcements coated in this manner.) (4) No porous materials, such as coke breeze or slag, should be used in connection with reinforced concrete work if such concrete is intended to be under water or exposed to the air. (5) Linseed oil or turpentine, or probably any other coating except cement or lime, applied to steel before its insertion in concrete, facilitates rather than prevents the rusting of the metal. (6) It is of great importance to ensure that the reinforcing steel is well embedded in the concrete, so that every portion is covered with cement. (7) The best results were obtained with aggregate consisting chiefly of broken stone or brickbats. Gravel would no doubt answer equally well.

Sea Water.—Whilst pure water is without chemical effect upon concrete, sea water, acting both chemically and mechanically, exercises an influence that must not be disregarded. To form some idea as to the probability of sea water affecting concrete, it is desirable to know its constituent parts. An average sample contains per 1,000 parts the following compounds or salts:

Sodium chloride	27·059
Magnesium chloride	3·666
Magnesium sulphate	2·296
Calcium sulphate	1·406
Potassium chloride	·766
Calcium carbonate	·033
Magnesium bromide	·029
Total salts	35·255
Water	964·745

In the foregoing list the magnesium and calcium chlorides and sulphates are the only compounds likely to have appreciable effect on the chemical nature of concrete; such action as they possess is due to the ions or acid radicles which they contain, and, although it is slight, the total quantity of these salts is so large that the effect on the concrete is much greater than the small proportion of them in the sea water suggests.

As regards the other salts present in sea water, the chief of which is common salt or sodium chloride, their action appears to be of but little consequence. In cases, however, where sea sand has been used in the gauging, the concrete takes longer to set, and it presents a damp surface for some time, due to the magnesium chloride and other hygroscopic salts in the sand, derived from the sea water, absorbing moisture from the atmosphere and so remaining damp.

A very practical contribution to the discussion of sea-resisting concrete is a Danish paper read before the Copenhagen meeting of the International Committee on Reinforced Concrete, by A. Poulsen, who gives the conclusions drawn from two series of experiments; the following is according to Dr. Desch's summary of the paper: (1) Good portland cements are very resistant to the action of sea water. No marked difference in the behaviour of cements slightly different in composition has been found, except that a high proportion of aluminates tends to cause disintegration. (2) In a dense mortar, the chemical action is confined to an outer layer of small depth, further action being checked by the slowness of diffusion. A porous mortar, by admitting salt water to the interior, is apt to crack by expansion owing to chemical change. (3) The main agency in the destruction of mortar and concrete in marine embankments, harbour works, groynes, etc., is not chemical action, but the alternations of saturation, drying in the sun, freezing, etc., due to the alternate exposure and covering by the rise and fall of the tide. Destruction takes place sometimes by cracking, sometimes by scaling, the latter effect being produced especially by frost. (4) The denser the mortar the better (1 cement to 3 sand is too poor for marine work). An admixture of fine sand with the ordinary sand increases the closeness of the mixture, but a well-graded aggregate is the most advantageous. (5) The

addition of finely-ground silica or trass to the cement before mixing is often advantageous in the case of the more porous mortars which are rich in cement. It is very doubtful whether anything is gained by adding trass to the denser mortars, which are rich in cement. (6) Hydraulic lime mixed with trass, etc., is of some value where a cheap material is required, in the mild climate and absence of tide of the Mediterranean, but it is incapable of withstanding the conditions of coast work in northern latitudes. (7) The destructive action of the sea being mainly physical and mechanical, and not chemical, tests by mere immersion in still sea water are of very little value in determining the behaviour of concrete in marine engineering works. A mixture which disintegrates under this test is certainly useless, but a mixture which passes the test may disintegrate under the more stringent conditions of practical use. (8) As long a period as is practical should be allowed for the hardening of concrete blocks before placing in the sea. The German custom of leaving them for one year in moist sand before setting in place is impracticable in many places, but should be approached as nearly as possible. (9) The behaviour of test-specimens for the first twelve months is very irregular, and definite conclusions can only be drawn from the results of long-period tests.

Acids.—Hydrochloric and sulphuric acids, while scarcely affecting siliceous materials such as sand and concrete aggregates, decompose the cement by dissolving out the soluble substances such as calcium carbonate, and they readily attack the aluminium compounds in the cement. Sulphates have a less powerful action than have chlorides, and they form products which are largely insoluble, and which, therefore, serve as protective coatings. Fortunately, as concrete consists largely of sand and aggregate. which are scarcely affected by ordinary acids, it offers a natural resistance to the common acids which would be liable to form by the action of chlorides or sulphates.

The action of hydrofluoric acid is one worth consideration, as, although a rare acid, its action is certainly of interest in special cases. The most characteristic property of this acid is its readiness to attack silica either in the combined state as silicates or in the free state as quartz sand. In fact, it is the only acid which will dissolve this substance. The acid may be formed in a variety of ways, such as by the interaction of the minerals fluorspar or cryolite and sulphuric acid, which produces—or at least releases—the powerful gaseous hydrofluoric acid, which is known to attack almost anything with which it comes into contact.

As has already been explained, small quantities of nitric acid are brought down by the rain, but larger quantities of nitrates are found in soils, where they are produced by the decomposition of vegetation and other nitrogenous matters; in cases where the concrete comes into actual contact with substances of this nature there is no doubt that in course of time it will be affected by the chemical action of the nitrates.

Many organic acids are without action upon concrete, but in the presence of sodium chloride (common salt) they are capable of dissolving the cementitious portion of it to a slight extent. Hence the action of saline solutions, such as sea water, on the cement present in concrete, may in some instances prove serious.

Alkalies.—Such alkalies as strong potassium hydrate (caustic potash) and sodium carbonate (soda) are capable of dissolving silica, a substance that is not acted upon by any acids except hydrofluoric.

Urine.—The action of this substance is of importance in view of the employment of concrete for stables, public conveniences, etc. Possibly, should concrete prove a durable material for such structures, a suitable surface finish will be evolved, so obviating the employment of the relatively costly tiling now in common employment. Urine contains water, urea, uric acid and certain organic and inorganic salts. Of these constituents the one most likely to influence substances of the nature of concrete is urea. By the action of moisture, urea $(NH_2)_2\ CO$, is turned into ammonium carbonate $(NH_4)_2\ CO_3$, which attacks silicates and substances of a siliceous nature, and hence tends to cause the decay of concrete in course of time. Uric acid is a weak acid, which is readily changed into urea by exposure to the atmosphere. The action of urine upon concrete would certainly be a slow one, but it calls for consideration where the durability of concrete in the construction of stables, etc., is concerned.

Waterproofing Concrete

THE permeability of concrete has proved a matter of great concern to many engineers and users of this material, especially in the construction of water tanks, sewers, aqueducts, gas-holders, etc., and numerous attempts have been made to render it impermeable to water, oil, and other liquids.

The methods used to render concrete waterproof are of two main classes (*a*) those in which a material of an impervious nature, or one which will combine with an ingredient in the concrete to form an impervious material, is applied to the surface of the finished concrete, and (*b*) those in which substances are mixed with the concrete so that the whole mass becomes impermeable. The latter method is, in many ways, the most effective, particularly if the surface of the concrete is likely to be damaged, as it gives the whole mass the maximum impermeability; but it is also more costly, and cannot be applied to the finished work.

SUPERFICIAL WATERPROOFING

Superficial waterproofing may be accomplished in a variety of ways, of which the chief are (*a*) coating the surface with an impervious material, such as cement grouting, dense mortar, tar, asphalt, or paraffin wax dissolved in light petroleum; or one which becomes impervious on drying—as sodium silicate (water glass) and some casein paints followed by formaldehyde or a solution of alum followed by a solution of soap, or soda lye, these alternate coatings being repeated as often as is considered necessary; and (*b*) applying to the surface a substance which will enter the pores in the concrete and will form with the cement an insoluble and impermeable compound. Sodium oxalate, which forms the insoluble oxalate of lime in the pores of the concrete, is typical of this class of waterproofing material, as is linoleic acid, which forms an insoluble lime soap; but the most largely used substances of this class are soluble silico fluorides, under the commercial term of *fluates*. These fluates are chiefly composed of aluminium, zinc or magnesium silico fluorides, and when applied to concrete they combine with the lime set free as the cement sets and, by forming double silico fluorides, they not only render the material waterproof, but also increase the hardness and durability of the surface to which they are applied. M. Merkuloff has shown that cubes composed of 1 part of portland cement to 3 parts of sand when immersed in a dilute fluate solution for several hours increased 50 per cent. in compressive strength, and that their resistance to the action of frost was similarly increased. The surface of the cubes was found to be ten times as resistant to abrasion as that of the untreated cubes.

MASS WATERPROOFING

By far the most effective means of rendering a mass of concrete waterproof is so to arrange the aggregate as to obtain a material practically devoid of pore spaces and with the closest possible texture. For poor concretes this may necessitate the addition of an exceedingly fine, insoluble powder, such as slaked lime or china clay. A careful grading of the aggregate and the sand used will produce a fairly close concrete, and then a pore filler is quite unnecessary, as has been proved over and over again. When pore fillers are employed, they should be added to the cement or concrete in a dry state, and be well mixed before water is added. Plastic clay is not a suitable pore-filler: it is too adhesive; but powdered china clay is suitable. Waxes, greases and oil are similarly inferior to china clay and to slaked lime. The only objection to slaked lime is its tendency to produce a whitish efflorescence on the surface of the concrete; china clay does not do this, but its slight plasticity is an almost equal drawback. If sand or calcined clay should be obtained in a state of fineness equal to that of china clay it would be almost perfect as a pore-filler, but, unfortunately, no grinding plant will produce such a fine product on a commercial scale at a price at which it can be used for this purpose.

As showing the differences in permeability

due to the use of equal weights of various substances in the same aggregate, the following table (due to *Concrete*) is interesting. The figures in the third column represent the number of cc. of water which penetrated concrete slabs 10 in. square and 3 in. thick, subjected to a water pressure on the face of 50 lb. per square inch during twenty-four hours. The slabs were taken from the moulds twenty-four hours after gauging, and were kept in cold water for twenty days before use.

Series	*3 per cent. waterproofing material added to cement*	*Permeability*
A	*Nil* . . .	1240
B	Mixture of slaked lime and lime soap .	1290
C	Resin	760
D	China clay . .	865
E	Equal parts resin and china clay .	740
F	Paraffin wax . .	1005
G	90 parts slaked lime, 10 parts paraffin wax	555
H	Slaked lime . .	450
I	Burned gault clay .	2370
J	Alumina . . .	320
K	90 parts china clay, 10 parts wax .	280
L	Equal parts china clay and alumina .	372
M	5 per cent. china clay	450

That the quantity of water used in gauging concrete has a large influence on the impermeability of the mass has been well established, but Cloyd M. Chapman has shown that the least absorption occurs when the concrete contains 13 to 15 per cent. of water, as shown in Fig. 476, which summarises the results of tests on 230 blocks.

The best method of obtaining a waterproof material thus appears to consist in using a carefully selected aggregate, together with the proper proportions of fine material and cement.

If the proportions of water, cement, sand, and aggregate are correct, and the aggregate is properly graded, there should be but little difficulty in obtaining a mass that is sufficiently waterproof for all practical purposes. The addition of a pore-filler to such a mass is unnecessary.

Water-repellent substances are also used in admixture with concrete, the idea being that a substance which is not merely insoluble, but actually repellent to water, must be more advantageous than such a substance as clay, silica, or alumina. Most of these water-repellent materials are of a fatty or oleaginous nature, oils, soaps, and the corresponding fatty acids, such as stearic and linoleic acid, being chiefly used. The use of blood, milk, and lard is extremely ancient, being recommended by Vitruvious. At the present time, mixtures of lime and tallow or of soap and alum are preferred.

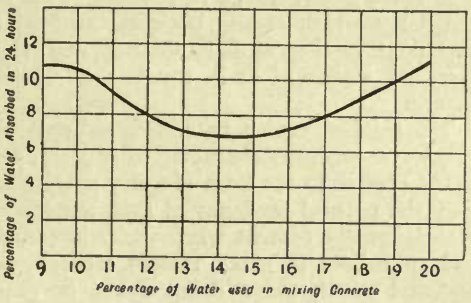


Fig. 467.—Graph Showing Relative Water Absorption of Concrete

These materials are usually supplied in the form of a powder which is mixed with the cement before gauging, but some workers prefer to keep solutions of the ingredients forming the water repellent, and to mix these solutions separately with the concrete. Thus, if a solution of alum is first added to the gauged material and then a solution of soda, there will be formed in the pores of the material a precipitate of aluminium hydroxide. Unfortunately, experiments have shown that this precipitate is very irregularly distributed, and that it does not increase the impermeability as much as the corresponding amount of finely divided alumina added to the concrete previous to the latter being mixed.

Emulsions, consisting of oil in the form of minute globules suspended in water, are also used to render concrete impervious to water. Their use is based on ignorance, as oils added to the mixing batch are more convenient, and the emulsion formed during the mixing process is better distributed through the concrete than when an emulsion is added in the first place.

Oils mixed with concrete, in the proportion of 10 to 15 per cent. of the weight of

the cement used, are used to render concrete waterproof. The oils used for this purpose must be of mineral origin, as animal and vegetable oils are liable to form acids which disintegrate the concrete. The most suitable for the purpose are mineral oils of various kinds, and range from heavy, black bituminous oils to light, non-volatile petroleum oils; white or colourless oils are used where the colour of the concrete is important.

It is unnecessary to add oil to good concrete; the addition will render it less waterproof and will reduce the strength.

A great disadvantage in the use of oil is the delay which occurs both in the initial and final setting and hardening and the loss of strength and toughness which ensues.

The chief objection to adding substances of a water-repellent character to concrete or to the cement in the form of a dry powder is that the natural tendency of such materials is to leave the cement when water is added and to collect in small masses, whereby a material of irregular composition is produced. The same objection holds good in the case of oleaginous and other substances which are mixed with water before use; the water merely carries the particles heterogeneously into the concrete mass, and does not effect a uniform distribution. Moreover, the use of materials which possess a strong water-repellent action undoubtedly tends to reduce the strength of the concrete with which they are mixed.

Extensive experiments with a large variety of substances (each of which was mixed with the cement in the proportion of 3 of substance to 100 of cement before use) show that water-repellent substances, such as oils, greases, soaps, resins, etc., have no true waterproofing qualities, and the slight benefit they confer is more than counterbalanced by the extent to which they weaken the concrete.

MAKING A NON-POROUS CONCRETE

The impermeability of concrete when obtained is due primarily to the absence of pores or voids in the material, and this can only be secured efficiently by a careful selection of material of suitable grades. For example, if sufficient cement is used and the mixing is thoroughly performed, all the pores into which water can penetrate will be filled. When, however, insufficient cement is used, a non-hydraulic material of equal fineness may be added to fill the pores. The amount of this material—which, as already indicated, may be of any convenient composition providing it does not attack the other constituents of the concrete—must be ascertained by experiment. The only satisfactory method consists in making a number of blocks with aggregates of various grades and to test the permeability or porosity of the blocks. The most satisfactory mixture is then made the basis of a further series of tests in which different percentages of pore-fillers are used. In this way, with care, it is usually possible to produce a perfectly waterproof concrete with little or no pore-filler in it.

Concretes that are easily penetrated by water and other neutral liquids will usually be found, on examination, to be composed of badly graded aggregates or to contain a considerable proportion of very coarse material. The larger the blocks the larger may be the size of the coarsest pieces of aggregate, but blocks of small size should not contain pieces of material which will not pass through a $\frac{1}{4}$-in. ring. The great difficulty experienced by some engineers in obtaining sufficiently waterproof concrete may usually be overcome by the use of material which will not pass through a $\frac{1}{2}$-in. or $\frac{3}{4}$-in. ring, and by screening out of the still coarser aggregate (if very large blocks are used) all matter in it which is less than $\frac{1}{4}$ in. in diameter. The use of too coarse an aggregate and of an aggregate containing too large a proportion of fine material are both equally detrimental to impermeability.

It is important to note that a concrets should not be judged too hastily as regarde its waterproof qualities. Until it is set all cement is porous, and many instances are known where the "weeping" of water-tanks ceased after the cement in the concrete had become properly hardened. Conversely, concrete which is subjected to the action of impure water (such as sea water or spring water highly charged with carbon dioxide gas) may be rendered permanently porous, whereas if it had been allowed to harden properly before being used it would have proved quite impervious.

It is frequently found that concrete that is insufficiently waterproof has been carelessly made or indifferently mixed. In such cases any supplementary waterproofing that may be necessary must be largely of a super-

ficial character, and will consist of the application of fluates, water glass, or some penetrable paint. The external application of ordinary paint is useless until the free lime in the concrete has been neutralised by one or more preliminary coats of a suitable acid. A coating of tar or asphalt is only available when the pressure of the water on it is negligible, as in buildings which are to be protected from rain, or floors which are liable to splashes. In such cases the tar or asphalt should be renewed as frequently as occasion requires, as its waterproof action is, of course, far from permanent.

COMMERCIAL WATERPROOFING SUBSTANCES

It may be helpful to explain briefly how some of the commercial waterproofing substances are added to the concrete. *Ceresit* is a white paste of butter-like consistency which requires to be mixed with an equal bulk of water and then with from 14 to 19 further parts of water. This is used instead of ordinary water for mixing the concrete (for cement mortar, 1 part of the special material to 10 of water is about right). The concrete must be made of a creamy consistence, so that it can be poured into place. *Hydrofuge-Castor* is a liquid bituminous substance which is incorporated with the mortar or concrete after the wet mixing has been completed in the ordinary way. *Pudlo* is a fine white powder which requires to be mixed with the cement before incorporating with any of the other materials. The water is added to the dry-mixed materials through a rose, to prevent the powder being lifted out of the cement.

Specifications, Quantities, Measuring, Estimating, and Pricing

The Specialist System.—Reinforced concrete, when first introduced, was in the hands of a few specialists, each firm having its own particular system and type of bar, the latter being generally patented. These firms tendered directly for the work, and undertook to supply the requisite drawings and calculations free of charge; on securing a contract, they prepared the necessary drawings, calculations, and quantities, which they submitted for estimates to one or more firms of builders licensed by them.

This system still obtains, but it has its objections—(*a*) The firm responsible for the design and superintendence of the work is commercially interested in it. (*b*) The system is inconvenient, inasmuch as it entails the employment of two contractors, whose work is interdependent, and each of whom may delay the work of the other. (*c*) It is not economical, as it adds an intermediate profit to the cost of the work, for, in spite of the specialist's claim that he supplies his design free of charge, it is obvious that he will only run his business as long as he makes a profit, and to do this he must either arrange a percentage to be included in the tenders of his licensed contractors, or charge sufficient for his steel to cover profit in addition to the large establishment charges entailed by the employment of a technical staff. (*d*) It necessitates the selection of a particular system and eliminates competition; but this objection has been partly met by the specialist submitting tenders from two or more of his licensed contractors.

As the new construction began to find favour, and came into more general use, architects, in their desire to obtain the best and most economical result, adopted the system of inviting two or more firms of specialists to submit schemes and estimates, and so placed themselves in the position of having to judge these schemes on two bases—namely, efficiency and economy—by striking some sort of a balance, and placing the schemes in order of merit; it is open to doubt whether many of the architects were in a position to take much practical account of the first factor, and whether the second factor was not usually dominant.

A GENERAL SPECIFICATION

It will be obvious that, assuming the contractors were left with a free hand (as they undoubtedly were in the earlier days), the competition was most unfair, as all of them were not competing on an equal basis. Therefore, with public departments, at any rate, it became the practice to issue with the skeleton drawings indicating the position and extent of the reinforced concrete work a specification which gradually became more and more definite and exact, specifying no detail which could apply to only one particular system, but setting forth all essential conditions and general principles with which all competitors must comply. Such a specification should fully describe the cement, sand, and aggregate; the composition of the concrete, and its method of mixing, whether by hand or machine, and, if the latter, the type of machine to be used; and the description and quality of the steel and its ultimate tensional strength. The centering should be specified to be truly laid with close joints, to be well strutted and braced, and made perfectly rigid and of sufficient strength to support the dead weight of the construction as a liquid, without deflection. The specification should mention any portions to be planed and coated with a wash of lime and clay to produce a smooth surface in any position where the concrete is not to be plastered, that the beam centering is to be designed to allow of the sides of the forms being taken down first, the minimum length of time to be allowed to elapse before the centering is removed, that it is not to be removed without the architect's consent, and the length of time the beams are to be supported after removing the remainder of the centering. A full description should be given of the tests that will be applied for the cement, steel, etc.,

and for the finished construction, stating the number and dimensions of the test pieces of steel and the percentage of area of slabs and beams to be tested, by whom the cost of the tests is to be borne, and the procedure in case they prove unsatisfactory.

Every condition which will ensure the calculations of all competitors being based as far as possible on the same data should be specified; to which end the maximum safe working stresses to be adopted for compression and shear in concrete, tension in steel (the tensional strength of the concrete being neglected), and for adhesion between concrete and steel should be clearly stated.

The foregoing outline is given, merely to emphasise the principal points to be attended to in drafting a specification for the purpose of obtaining combined schemes and estimates in competition. It should be written in as much detail as possible, and embody every condition capable of general application, such as accessibility of site, means of getting in and hoisting materials to the various levels, provision of sheds for storage, etc. (unless these are to be provided by the general contractor, in which case the fact should be stated), protection of work from frost, cessation of work during frost, whether the contractor is to supply water or whether this will be supplied by the building contractor, whether he is to supply all his own plant and hoisting tackle, or if and to what extent he is at liberty to use that of the building contractor, and what special plant he must provide, etc., etc. The aim must be to ensure that all competitors shall tender on equal terms, so far as this is possible where each competitor is submitting a different scheme.

It is also advisable to specify that the work must be carried on concurrently with that of other contractors, who are to be allowed all reasonable facilities for carrying on their work, and to state the approximate date at which the various portions of the building will be ready to receive the reinforced concrete, but that the building owner will not hold himself responsible for any alteration in those dates due to delay on the part of the other contractors. A corresponding clause should be inserted in the building contractor's contract specifying these dates, and also indemnifying the building owner against any claim for delay due to the reinforced concrete contractors. A clause should be inserted in each contract stating that no claim will be entertained for damage to plant, materials, etc., caused by other contractors using the site.

In the absence of such precautions, claims for delay from either contractor will lie against the building owner, whose sole remedy is, in turn, to prefer a claim against the offending contractor, the result of which is by no means as certain as is the necessity for meeting the costs of the action.

The contract under the foregoing conditions being for a lump sum, it is very necessary to attach to the specification a schedule of prices for measured work to be adopted in variations, and prices of materials and labour rates to be adopted in day work; these prices (except the labour rates) may be attached by the surveyor, and thus become practically a condition of contract, or they may be left to be priced by the contractors, in which case they should be carefully examined before accepting any tender. The labour rates should always be fixed by the surveyor, and should be the net current rates payable in the district. The percentage that will be added to these to cover contractor's profit, superintendence and "sharps" (allowance for sharpening tools) should be definitely stated.

From skeleton drawings and a specification as outlined above, the competing firms prepare their detailed schemes and estimates. Public departments and some private architects require calculations to be submitted with the estimates, a good practice when the architect can check them.

QUANTITIES

In order to frame an estmate under such conditions the reinforced concrete specialists have to prepare their own quantities, and this is usually done in the most rough and ready way. Not only are items of differing values lumped together, but it is open to question whether the actual amounts will bear investigation. All the concrete is billed per yard cube, even slabs 3½ in. or 4 in. thick; the centering is lumped together, little or no differentiation being made between differing values; in some cases even the centering for beams and slabs is lumped together, no height being stated for the strutting and bracing, and the steel reinforcement billed together in one item for each level, or even in one item for all levels. These quantities are supplied to the licensed firm or firms on which to estimate for the

actual work, and it will generally be found that the concrete is priced at one average price per yard cube, and the centering and steel each at an average price per yard super and per ton respectively; such averages are rough and not exact. It has been argued that as reinforced concrete is a comparatively new material, it is useless to supply any more detailed quantities, as the builders have not yet had sufficient experience to price them accurately. This contention referred particularly to the centering, the one item that even at an early date could have been priced with reasonable accuracy if properly measured; in any case, now that this mode of construction is becoming so general the argument no longer holds good, and there is no excuse for continuing these unsatisfactory methods.

As long as the present system of obtaining schemes and estimates continues, it will be impossible to supply *complete* bills of quantities, because, obviously, a scheme must be prepared before quantities can be taken off. Attempts have been made to prepare bills of quantities for the purpose of obtaining schemes and estimates with a view to greater accuracy and uniformity of practice than at present prevails, and, though their practical utility may be open to question, instructions will be given in this chapter for their preparation. The bills would be supplied to competitors, together with a set of skeleton drawings and a specification as previously described, on which to base the scheme; or, better still, the specification might be incorporated in the preamble to the bill.

Such a bill would necessarily be in skeleton form, and should, like the specification, be in as much detail and as complete as it is possible to make it in the circumstances. The preliminary bill should contain the requisite data for preparing the scheme; it should, in fact, be a complete specification, items which carry a money value being written in bill form, as for example: "Provide a close boarded weather-proof shed, with floor raised above ground, having a floor area of — ft. super, for the storage of portland cement, and clear away at completion"; where two contractors are employed, this item would probably be in the general contractor's contract.

Procedure on the Job.—Before discussing the measured items of this bill, it will be advisable to consider the usual mode of procedure in carrying out a typical reinforced concrete job, as this will govern the descriptions, particularly in respect of the centering. The centering for the lowest floor is erected first, the reinforcing steel placed in position, and the concrete deposited, tamped in, and levelled. When it has set sufficiently, sole plates are placed in position to distribute the weight, and the centering for the next floor is erected on them, the steel placed in position, the concrete deposited, and the whole process repeated for the next floor, and so on, care being taken to place the uprights and struts for the centering of each floor as nearly as possible over those on the floor below. Any deviation from this order, by increasing the height and scantlings of the posts and struts, and possibly making the supply of materials to the lower levels more difficult, may tend to make the work more costly.

Preparing the Quantities.—The usual practice of the specialist firms in preparing their quantities has been to keep the whole of the work, centering, steel and concrete separate for each level, this being a convenient method, which is correct in principle. In compiling a bill of this description, it must be borne in mind that, in addition to the actual quantity, the description must impart sufficient information on which to base the calculations; for example, to determine the thickness of a slab and the amount of reinforcement, it is not sufficient to give the span only, because slabs "fixed" at edges, or, what amounts to the same thing, continuous over supports, are stronger than those supported only; and slabs supported on one edge and fixed at the other hold an intermediate place between the two kinds. Top reinforcement must be provided over the supports in all cases where the slabs are continuous. Also, rectangular slabs, supported or built in on four edges, may be reinforced in both directions; all of which information must be in the possession of the designer before he can fill up the blanks left for him in the bill of quantities. Where the positions of the beams have been determined beforehand and indicated on the skeleton drawings, the dimensions of the beams and their reinforcement only being left to the designer, it is possible to embody the essential conditions in the description; for example, in the accompanying sketch (Fig. 477) of a floor slab over a room or

apartment 45 ft. long by 20 ft. wide, supported on two beams across the width of the room, the centre bay will be continuous over both supports, the end bays continuous over one support, and the edges in walls supported only; in this case the slab might be billed in two separate items, each with its proper description; but it would convey a much better idea of the requirements of the case to the mind of the designer if the whole slab were combined in one intelligible item, the description reading thus: "Floor slab at first floor level over an apartment 45 ft. long by 20 ft. wide, the length divided into three equal rectangular bays, and the slab continuous over the two intermediate supports."

Nothing less than the above description can, in the absence of illustration, convey the necessary data.

A much simpler and at the same time more efficient method of conveying the information is by lettering or numbering the various positions on the skeleton plans and referring to them by corresponding letters or numbers in the quantities. The above description would then be curtailed to "floor slab at 'A' at first floor level"; this method enables the competitor to design his work from the skeleton drawings, and then to trace, without a moment's hesitation, the various items in the quantities and complete and price them.

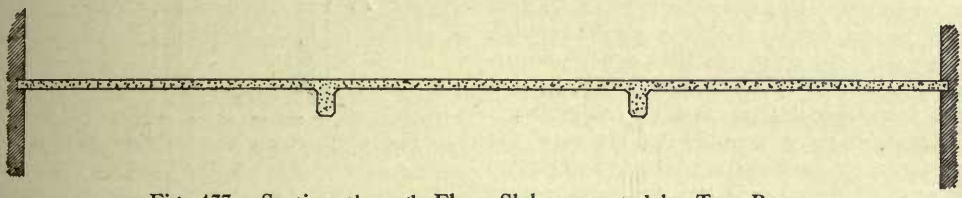


Fig. 477.—Section through Floor Slab supported by Two Beams

GENERAL OUTLINE OF BILL OR SCHEDULE

In "taking off," the most convenient order to adopt will be: concrete in slabs, reinforcement to ditto, centering to ditto, concrete in beams, reinforcement in ditto, centering to ditto. One room or apartment should be completed before starting the next. Walls and columns should be taken off in similar order, being separated into different floor levels, and their heights stated. External and internal walls should be kept separate.

The order in the bill at each level may conveniently be similar, the various slabs, walls, columns, beams, etc., being kept separate under the different reference numbers or letters (their sizes being unknown to the surveyor).

Slabs.—The reinforcement for the slabs being billed by weight may be in one item at each level, described as "steel reinforcement to slabs," and a blank left for the amount.

The centering to the slabs is billed in squares or yards super, the height of the strutting and bracing stated and the different heights kept separate.

Beams.—The concrete in beams is billed in yards cube, and may, therefore, be in one item at each level; the word "cube" should be inserted in the proper column, the description written, and a blank left for the amount.

The steel reinforcement to beams is described as such and billed by weight, a blank being left for the amount.

The sizes of the beams being undetermined, the girth of the beam centering is unknown to the surveyor, and must therefore be billed per foot run and described as "extra materials and labour over slab centering to casing to beams, — in. on soffits, and — in. deep," blanks being left for sizes; the different heights for strutting and bracing are kept separate.

Walls.—Concrete in walls should be billed in yards super up to 12 in. in thickness, the height of the wall from its floor level and the height of floor level stated; blanks must be left in the description for the thickness. The steel and centering will follow in the usual order, the description given with the concrete conveying the necessary information for computing the height of hoisting the steel and the amount of strutting to the centering.

Columns.—Concrete in columns is billed in yards cube, and heights stated as in the case of walls; the word "cube" must be inserted in the proper column, and a blank left for the amount. The steel and centering will follow in the usual order.

Where it is decided to strengthen a wall by means of piers, the concrete in them should be billed in yards cube, the description stating that it is "in wall piers"; the steel and centering will follow as usual. The size of the piers being undetermined, the centering will have to be billed in feet run as in the case of beams, and described as "extra materials and labour over centering to flat wall surface for casing to wall piers, — in. wide on face, and — in. projection," blanks being left for the sizes. Where the arrangement and number of piers have not been settled it will also be necessary to leave a blank for the amount.

Centering.—There is some diversity of opinion as to the amount of detail in which centering should be measured, some surveyors contending that all extra labours, such as the intersections of beams, should be measured; others measure only the superficial quantity of centering and expect the price to cover all incidental labours. There can be no doubt that the more detailed method is the only one that can be accurately priced, and that the less detail supplied the more speculative must be the pricing.

It is scarcely possible to measure the centering for combined schemes and estimates in as much detail as though the scheme were already designed; the most practical method of measuring for this purpose is to keep the items of obviously differing values separate, and to include all labours which cannot be measured in the descriptions, which should be as definite as possible. To avoid undue repetition, the method of measurement should be clearly stated in the preamble by clauses such as the following: "All items of centering are to include the use and waste of material, all labours in erecting and removing, and all cutting and fitting at angles and intersections. The measurements have been taken net as erected."

It would serve no practical purpose to attempt a list of centerings for various positions classified according to their value, but taking the simplest descriptions of two classes, namely, to plain slabs and to rectangular beams, it should be a simple matter to separate and describe others which may deviate from them in any particular.

Raking, cutting, and waste should be measured lineal. Where it is specified that all internal and external angles are to be finished with splays 2 in. wide (or as the case may be), these items should be treated as extras on the plain centering, and described as "extra labour forming splayed internal angle 2 in. wide, between wall and slab centering," or "between beam and slab centering," as the case may be, the principal reason for the distinction being that the centering at sides of beams is usually specified to be removable without disturbing the rest of the centering, in which case the splay would be removable with the side of the beam. It must not be forgotten that in all cases of "extras" the original measurement must include the whole surface; for example, in Fig. 478 the beam centering would be measured the length by the girth from A to A, and then a lineal dimension would be taken on both sides of the beam and described as "extra on centering to beam for forming splayed internal angle

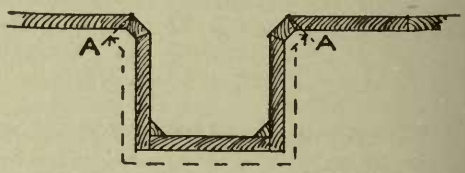


Fig. 478.—Cross Section through Beam Centering

2 in. wide"; to form the splay on the external angle an angle fillet would be fixed in the internal angle of the centering as illustrated. This should be measured and billed per foot run, and described as "splayed angle fillet 2 in. wide on the splay, and fixing in angles of beam centering to form splay on the external angle of the concrete." Other instances where items occur which might with advantage be measured as "extras" will be readily distinguished as they present themselves.

In the case of beams with curved soffits, the soffits will, of course, have to be measured separately from the sides, but the two should be added together and billed in one item, and described as centering to beams, semicircular, segmental, elliptical, or as the case may be, on soffits.

Not only the centering, but the concrete and reinforcement should be separated into classes of differing values, for, in addition to convenience and the avoidance of repetition in the descriptions due to the three items following one another in each class, there is the fact that the more complex forms will entail more labour in placing and tamping

the concrete, and more elaborate reinforcement, as well as more expensive centering.

A Detailed Bill Advisable.—Seeing that labour is the great factor in the cost of reinforced concrete, and that the amount of labour must vary considerably in the different classes of work, it is scarcely logical to argue that it is waste of time to differentiate between them, and that the contractor can price what is known as a "lumped" bill with as much accuracy as he can a classified or more detailed one. Although these more or less rule-of-thumb methods may have obtained in the past, there can be little doubt that, with the more general adoption of reinforced concrete construction, and the keener competition as firms of general building contractors in increasing numbers become skilled in its use, every opportunity will be taken of pricing more accurately, and consequently in more detail; in short, a properly prepared bill of quantities for reinforced work will be priced with as much care as one for ordinary building work is at present.

Such, then, are the general outlines of a special bill—or, as it might be more correctly styled, schedule—prepared before the reinforced concrete construction is actually designed, for the purpose of obtaining schemes and estimates for different systems. Such a bill would undoubtedly ensure the estimate being made in a certain amount of detail and the items being more or less correctly measured. The bill would necessarily contain a considerable amount of repetition—for instance, all the 4-in. slabs could not find their way into one item; all the descriptions would be written by the quantity surveyor, but of the actual quantities, some would be supplied by him and some by the contractor, thus entailing divided responsibility, which would be most undesirable. Moreover, it must be obvious, from what has already been said, that in some cases, more especially those of complete buildings including walls, so little detail can be actually known of the completed scheme that it would be almost a farce to attempt to measure anything beforehand.

An Alternative to the Bill.—On the whole, it is open to question whether the preparation of a bill under these conditions will pay for the trouble involved, and whether it is not more reasonable to supply a specification containing a short schedule of instructions, naming the items to be measured and their method of measurement, and attaching the condition that, before any tender is definitely accepted, the contractor will be required to submit his tender on such schedule priced in detail.

To give an example of a schedule here, or to enter more fully into the details of a bill, would be premature, as full instructions for the preparation of a bill from a complete set of drawings and specification are given later. From these, in conjunction with the foregoing instructions, there should be no difficulty in preparing a bill or schedule for any given set of conditions.

"One Contractor" Jobs.—It will be conceded that it is a great advantage to have the whole of a big job carried out by one contractor experienced in reinforced concrete as well as in ordinary construction. This avoids any division of the responsibility, and considerable economy both of time and money results. So many firms now are prepared to do this that there is no difficulty in the matter; several have "systems" of their own, others are licensees of one or other of the well-known firms of specialists, and all would doubtless be prepared to carry out any special system selected or a scheme specially designed by the architect or his engineer.

In all cases where the particular system to be adopted has been decided beforehand the work can, of course, be properly measured and incorporated in a bill of quantities, and if it is decided to invite only builders prepared to do the whole of the work, then one bill of quantities will suffice; but the whole of the reinforced concrete work, including the steel and centering, should be billed as a separate trade, and not split up and embodied in the ordinary trades of "concretor," "carpenter and joiner," and "smith and founder."

DESIGNS INDEPENDENT OF THE SPECIALIST

The ideal method of dealing with reinforced concrete work is for the architect to design his own or employ an independent engineer to do so, being then free to use any section of bar considered best for his particular purpose, or any of the patent bars (provided the patentees would supply them), or even to use two or more different bars in the same building, choosing them for their fitness for particular positions and uses. Some, at any rate, of the patentees are

already prepared to supply their steel under these conditions, and if the system became in any degree general all would be obliged to fall into line.

The great obstacle to the adoption of such a system is the fact that comparatively few architects can design reinforced concrete work—nor can they reasonably be expected to do so, as it is purely engineering work, foreign to the architect's nature; why, however, an architect should not employ an independent engineer, instead of placing himself in the hands of commercial firms, is not quite so easy to see, except for the difficulty of explaining the situation to the client, and inducing him to pay the fee, and the reluctance of the architect to paying it out of his own pocket. It cannot be too strongly impressed on clients, however, that the fee must be paid in any case, either as such or as extra price of materials. For example, take a case where reinforced concrete is used for all floors, beams and flats in which the total cost of the reinforced work, exclusive of all ordinary builders' work, is £1,000; assume that 450 cwt. of steel (a reasonable average case) will be employed. This steel would be supplied to the builder at, say, £14 per ton (this is one of the actual rates), to which he would have to add the cost of hoisting and placing in position and profit, an average of 25s. to 30s. per ton. Now, £8 per ton would be a very good price for ordinary steel rods, or, allowing for extra labour in bending and fixing stirrups, say £9. The addition of £1 10s. to this ought to be sufficient to cover the extra value of any patent bar. Assuming, then, for the sake of argument, £10 10s. as a proper market price for the patent bar, instead of £14, a surplus of £3 10s. per ton, or a total of £78 15s., equivalent to 7⅞ per cent. on the total cost of the reinforced concrete work including the centering, will remain to pay for drawings, superintendence, etc., by an independent engineer.

Reinforced concrete being as much a mode of building construction as is a system of brick walls with wood or steel joists, it is difficult to believe that architects will continue to ignore its study; in any case, there can be no reason why public departments should not see that a section of their large technical staffs is skilled in this branch of work, and so make themselves independent of commercial firms as far as design and calculations are concerned.

MEASURING

For the purpose of measuring and estimating, at any rate, it will be necessary to assume a settled and complete design before the system of measurement can be explained.

There are many instances, particularly, perhaps, in England, where reinforced concrete construction is favoured for floors, flats, roofs, partitions, columns, and beams, where its use for external walls would not be considered. In such cases there will be several items of builders' work in connection with the reinforced concrete; where there are two contractors these items will be executed by the building contractor, where only one contractor is employed these items must still be measured, but should be billed in their proper place in the ordinary trades, and not under the heading of "reinforced concrete"; these items include such as "cutting or leaving chase — in. deep for edges of floor slabs and making good," "cutting or leaving recesses for ends of reinforced concrete beams and padstones (here state dimensions), and building in in cement," etc., etc.; similar items occur in the case of ordinary concrete and steel joist construction, and therefore they will not be further referred to here, it being assumed that the reader is already acquainted with the modes of measurement of ordinary builder's work.

As already stated, it is the practice of specialists, when taking their own quantities, to measure all concrete cube; while this practice is here deprecated, no undue or unnecessary elaboration will be advocated, but the builder should be given the opportunity of judging what class of work he is estimating for. The fact of a contractor pricing two or more separate items at the same rate is not necessarily a criticism on the work of the surveyor; the contractor should be given the opportunity of judging their relative values for himself, and it is not a good practice for the surveyor to do it for him. Again, the contractor may not have considered these items of equal value, but may have priced them at an average value after noting the ratio between the quantities of the different descriptions, in which case, assuming a variation on the contract entailing an addition to the more expensive and an omission on the cheaper description, the contractor would, under the usual terms of a building agreement, stand to lose, as all variations would be valued at

the rate stated in the bills; he has, however, been provided with the requisite data to enable him either to price separately, or to form a true average, whereas, when items of differing values are combined under one description, the contractor has no option but to affix an average rate, and a speculative one at that.

There is every reason to suppose that reasonably detailed quantities and increasing competition will gradually lead to more careful and accurate pricing.

Measuring Concrete.—The concrete in walls and partitions up to 12 in. thick should be measured super, also rafts up to 12 in. in thickness. Over this thickness, these items should be measured cube; concrete in foundations, columns, piers and beams should also be measured cube. The work should be kept separate at the different levels; floor levels will form convenient divisions, but their height above ground should be stated. The different classes of work should be kept separate; for example, slabs on the slope (as to pitch roofs, etc.) should be kept separate from horizontal slabs. Note that slabs laid to a pitch of 45° or less may be laid *on* single centering, but that to steeper pitches the concrete will have to be packed *between* centering; the two classes of work must be kept separate as the labour is different. The descriptions of the two classes respectively would be somewhat as follows: "5 in. concrete slab laid sloping," and "5 in. concrete slab sloping and packed between centering." Arched beams should be kept separate from straight ditto; circular walls and partitions separate from straight ditto, and the radius stated. Vaults should be measured super, stating whether segmental, semicircular, etc.; and thickness given. Domes should be measured super, stating the thickness, unless this varies, in which case it will be necessary to measure them cube, stating that it is in domes, the thickness varying from — to — (the superficial area of a dome is the same as that of the circular portion of a cylinder exactly enclosing it). Concrete in curved ribs should be measured cube, and described as in semicircular or segmental ribs, or as the case may be, and the radius stated. The extra concrete forming internal splays to angles may be measured either per foot run and sizes given, or per foot cube and described; the latter method is preferable, as the concrete is only worth the same rate as that in the wall or beam to which it is attached, the extra labour being on the centering.

In measuring concrete, no deduction is made for the space occupied by the reinforcement.

Small Holes.—Small holes required to be left in the concrete for the passage of pipes, etc., should be numbered, and their size given, the wood plugs or drums fixed on the centering being included in the description of the holes.

Every precaution should be taken against having to cut into the concrete after it is laid. It is better and cheaper to bed bolts with back plates in the concrete, and leave them projecting the requisite distance for fixing. These bolts have to be fixed in position in the centering. The bolts themselves are usually provided under the building contract, but they must be numbered for the labour of "bedding in." To *cut* a hole for a bolt or any other purpose in reinforced concrete is an expensive item of labour, the material being so very hard.

Measuring Steel Reinforcement.—The steel reinforcement is measured per foot run and billed in cwt., qr. and lb., billing to the nearest 7 lb.; for example, 54 lb. would be billed as 2 qr.; 60 lb. as 2 qr. 7 lb.; 59 lb. as 2 qr.; 73 lb. as 2 qr. 14 lb., and so on.

Care must be taken in measuring the steel to add for all laps and, unless these are clearly shown on the details, the necessary information must be obtained from the designer. The weights of the bars per foot run may, in the case of standard bars with rigidly attached web members, be obtained from the trade catalogues; in the case of loose stirrups, these will have to be separately measured and weighted out; they should be measured from an actual sample bar, in order to obtain the correct length around the bends or twists; they may then be weighted out from tables of weights or some sample 12 in. pieces actually weighted; in the case of standard stirrups of any particular system the weights may either be obtained from the trade catalogue or by actual weighing; stirrups may either be measured per foot run or numbered, whichever lends itself the more readily to the subsequent weighting out; in the case of bars and rods it is only necessary to remember the weight per foot super of 1 in. plate, and the weight per foot run of 1 in. diameter rods to obtain the required weight of any

other sizes; for example, the weight of 1 in. by 1 in. bar per foot run will be $\frac{1}{12}$th that of 1 in. plate per foot super; that of 1 in. by 2 in. $\frac{1}{6}$th; that of 1 in. by $\frac{1}{2}$ in. $\frac{1}{24}$th and so on; in the case of rods, seeing that the areas of circles vary as the squares of their diameters, it will only be necessary to multiply the weight of 1 in. rod by the square of the diameter of any other size to find its weight; thus the weight of a $\frac{1}{2}$ in. diameter rod will be $\frac{1}{4}$th that of 1 in.; that of a 2 in. rod four times that of 1 in.; and so on.

The steel reinforcement should be kept separate for the different positions and classes of work. In a job where bars with fixed web members, such as the Kahn, are used and it is necessary in certain portions of the work to use loose stirrups, this work should be billed separately and properly described, there being considerably more labour in placing steel bars and loose stirrups than in placing a bar having fixed stirrups.

Measuring Centering.—Centering is not only a very important item, but also a very costly one. Moreover, the value of the different classes of work varies so considerably and extra labour forms such an appreciable item that it should be measured as carefully and in as much detail as the circumstances of the case permit. There have probably been more surprises at the end of a job due to the speculative nature of the items of centering than to any other cause.

"Use and Waste."—The concrete and the steel can be priced with reasonable accuracy, even when they are not taken in as much detail as they should be, but not so the centering. That must be, unless carefully measured in detail, a very speculative item. The materials are charged as use and waste only, but the centering has to be very carefully erected, and labour enters largely into the cost. No definite proportion can be fixed for "use and waste," as this differs so widely in different classes of work; in the case of centering to a plain floor slab, there will be a certain amount of waste in cutting to the required lengths, but nearly the whole can be re-used by further cutting on another job, or possibly in another position on the same job. The larger the areas of the slabs the less waste will there be. The labour, also, in erecting slab centering is comparatively simple. But in beam centering there will be much more cutting and waste and more labour. As the dimensions and span of beams decrease the waste and labour increase; in elaborate works, much of the centering will scarcely be worth the cost of carting from the site, and though still technically described as "use and waste," its full price ought to be included in the estimate.

First-class Centering.—Readers will know that a perfectly true and finished surface on walling and mouldings, and even on carving (so called), provided there is no undercutting, can be left direct from the centering or moulds without any subsequent rendering; but for this purpose the falsework must be wrought on one side and jointed up as a first-class piece of joinery, the carved work being cut in reverse in wood blocks, which are inserted in their proper position in the wall centering. Such centering will, of course, be far more expensive than the ordinary style, and more labour and care will be entailed in packing the concrete.

Centering required to be planed and covered with a wash of lime and clay to leave a smooth surface on the concrete may be kept separate and described, but it is preferable to measure the planing and coating separately.

In order to form a key for plastered ceilings, it is sometimes specified that a layer of fine breeze is to be spread on the slab centering before filling in the concrete, this should be measured separately and billed in yards super.

Centering for a scheme already designed can and therefore should be measured in more detail than when measuring for combined schemes and estimates.

"Centering" Clauses in Preamble.—In the case of ordinary building works, it will often be impossible to separately measure the bearers, struts, and braces to the centering, as it is unusual to supply drawings for this, but any clauses in the specification governing the design and erection of the centering should find their way into the preamble. In the absence of any more definite specification, some such clause as the following should be inserted in the preamble: "The centering to be well strutted and braced and made perfectly rigid and strong enough to sustain the weight of the slabs, etc., as a liquid mass, without deflection." This clause, used alone, throws the responsibility for the efficiency of the centering on the contractor. To avoid

repetition in the descriptions, the following clause may be inserted in the preamble: "The centering to include all bearers, struts, braces, etc., wedging, and all ironwork required unless otherwise stated"; it will then only remain to state the purpose of the centering and the height of the strutting in the description.

Specially Designed Centering.—Where, as in the case of heavy engineering works, or from the special nature of the work, the engineer or architect elects to design his own centering it will be advisable to measure it in detail as designed, otherwise it would be difficult to enforce the work being carried out to drawings without incurring an extra. When measuring strutting and bracing in detail, it is advisable to measure the timbers lineal and to state their scantlings, rather than to measure them cube as in the case of ordinary carpentry. The architect's permission should be obtained to insert a clause in the preamble to the effect that the scantlings given for the timbers to the centering are minimum scantlings, but that the contractor is at liberty to use other scantlings of equal or greater strength with the architect's approval, provided no extra cost is incurred. The object of this clause is to enable the contractor to use such scantlings as he may have in stock or as are most easily procurable, so as not to increase unnecessarily the cost of the temporary work.

Centering should be measured super (except as hereafter stated) and described, and the height of the strutting given; that at different levels and to different descriptions of work should be kept separate.

Openings.—All openings should be deducted where it is practicable and convenient to omit the centering, but in the case of small openings, particularly in horizontal slabs, it will often be cheaper to run the centering through than to provide extra supports or trim the bearers round the opening, in which case it should not be deducted, but the vertical form to edges should be erected on it. It will be necessary to keep some openings clear for access and working purposes; staircase and lift openings are usually the most convenient for this purpose in the horizontal slabs, and door openings in the walls. In all cases where the centering is deducted, an item should be taken of "cutting and waste around openings"; with regard to doors and windows these may be numbered and the average size of openings given, stating whether one or both sides are measured, or the dimension may be taken run, and a similar dimension taken on slabs around openings for stairs, skylights, etc., etc. Keep the centering to edges of openings in narrow widths separate.

The vertical centering around openings in slabs should be described as such, and measured super; where 9 in. or less in height, it may be measured run and the height given.

Centering to Walls.—Vertical centering to walls and partitions should be measured to both sides; it is not a good practice to measure one side only, stating in the description that "one side is measured for two," because the amount often varies considerably on the two sides of a wall, especially if it be an external wall with cross walls abutting against one side only.

Centering to external walls should be kept separate. There are not quite the same facilities for getting at both sides of the centering and the upper stories can only be strutted from one side.

Moulds for Architectural Features.—Where centering has to be formed into moulds for architectural features it should be kept separate, as more labour and waste is involved. It may either be separately measured and the vertical wall centering behind it deducted or measured as extra on the vertical centering; in most instances, the latter method is to be preferred. A typical case is that of a shaped core for a moulded cornice to be finished in cement, the simplest and probably the best method of measuring which is per foot run as extra material and labour over vertical wall centering, stating in the description both the height and the girth of the centering, so as to enable the estimator to ascertain exactly the extra net amount of material involved and to judge the extra labour and waste. By making it a lineal dimension, there is also the added advantage that the mitres, etc., may be "written short" in the bill. It should be particularly noted that this item is described as extra *materials and labour;* this makes it perfectly clear that no deduction has been made from the vertical centering for that portion immediately behind the cornice; nor, on the other hand, has any material been previously measured to cover the projection.

Centering to Inclined Slabs.—Centering

to inclined slabs (as for pitched roofs, etc.) should be kept separate from horizontal slabs. Centering to slabs inclined at a steep pitch (more than 45°) requiring centering both sides should be kept separate; in this case the description must be carefully worded to avoid any ambiguity, thus: "Centering to both sides of inclined slabs (both sides measured)."

Centering to Beams.—Centering to beams should be measured super and the different descriptions of beams kept separate; as previously explained, the super is obtained by taking the length by the girth of the centering on the outside of the sheeting. Internal and external chamfers on beams, etc., should be measured as has been previously explained on p. 274. Intersections of beam centering with beam and of beam centering with wall or padstone centering should be measured where the end of one beam abuts on another beam, padstone or concrete wall. A piece has to be accurately cut out of the centering of one of the beams or the wall, and the end of the abutting centering accurately and closely fitted thereto. The most comprehensive way of measuring these items is to number and describe them, stating the girth of the centering of the abutting beam; see example below:—

cleanly from the concrete should be either included in the description or separately measured.

Moulds for Cornices, etc.—The moulds for cornices, string courses, etc., should be measured lineal as extra on the vertical centering, internal and external mitres, and mitred and returned ends numbered and kept separate. Any ends fitted against plain centering should also be numbered, such as an internal angle where the cornice is not returned, in which case the vertical centering would have to be cut and scribed to the outside of the cornice moulds; these numbered items should be "written short" in the bill. Blocks for carving should be numbered and described, and the size stated. The actual carving of the blocks must be treated as a labour item, and is best provided for by a provisional sum, unless it is of a geometrical or other simple nature such as could be dealt with by a description or sketch. The typical example on the next page will serve to illustrate the descriptions and method of billing. The reference letter "A," in addition to the purpose it serves in this section of the bill, will be found even more useful in saving repetition in the descriptions of similar centering at other levels.

						£	s.	d.
		No.	3	Intersections of beam centering 27 in. girth				
		,,	5	Ditto ditto with wall centering 30 in. girth				
		,,	4	Ditto ditto with padstone centering 24 in. girth				

Centering to Padstones.—This should be measured super and kept separate.

Centering to Columns and Piers.—This should be measured super, each description, such as "to columns," "to piers," and "to wall piers," etc., being kept separate and properly detailed. They should be billed in their proper order of floor levels, and their heights stated. All junctions of columns or piers with slab centering should be numbered and their girth stated.

Centering to Produce Fine Finish.—Centering, such as that described on p. 278 to produce a finished face on the concrete direct from the moulds, should of course be kept separate and carefully described; any coating of the centering or other device to allow of the centering coming away

ABSTRACTING

In many cases an abstract will scarcely be necessary, but in others it may be found more convenient to have one. It is assumed that the reader is acquainted with the methods of measuring, abstracting and billing ordinary builder's work, and it will therefore be unnecessary to do more here than to suggest a convenient form for abstracting reinforced concrete work. A perfect abstract should present the items in the correct order for billing; it has been stated before that it is the custom of specialists to group the three main items of concrete, steel, and centering in sections according to levels. Floor levels are adopted as far as possible, as being convenient divisions. This method, although preventing all the

Yds.	Ft.	In.			£	s.	d.
200	—	—	Sup.	Centering to outer face of external 9-in. walls from first to second floor levels, the boards tongued and grooved and carefully jointed and clamped, the inner face to be planed to a true and even surface, so as to leave no marks of this jointing in the surface of the concrete			
A							
	100	—	Run	Extra materials and labour over vertical centering as "A" for centering to moulded cornice 24 in. girth 12 in. high as sketch, carefully jointed to the vertical centering; all joints to be close and to be grooved and tongued, and the internal surface of the mould clean and true, so as to leave no markings on the concrete			
				No. 5 Internal mitre			
				No. 10 External mitre			
				No. 2 Mitred and returned ends			
				No. 1 End fitted to vertical centering and the vertical centering cut and scribed			
				No. 3 3 in. blocks, size 2 ft. 0 in. by 3 ft. 6 in., carefully jointed up and prepared for carver, and cutting out for and inserting and carefully jointing same to vertical centering as "A." (Vertical centering not deducted.)			

items of either material being billed consecutively, places each in its proper relation to the others in a very comprehensive manner, and is to be commended for the main items. It is advisable for the sake of clearness to apply the system to the main items only, and to bill all incidental items and labours under a further sub-heading of "Sundries," which will comprise more particularly such items as are not affected by the level at which they are executed.

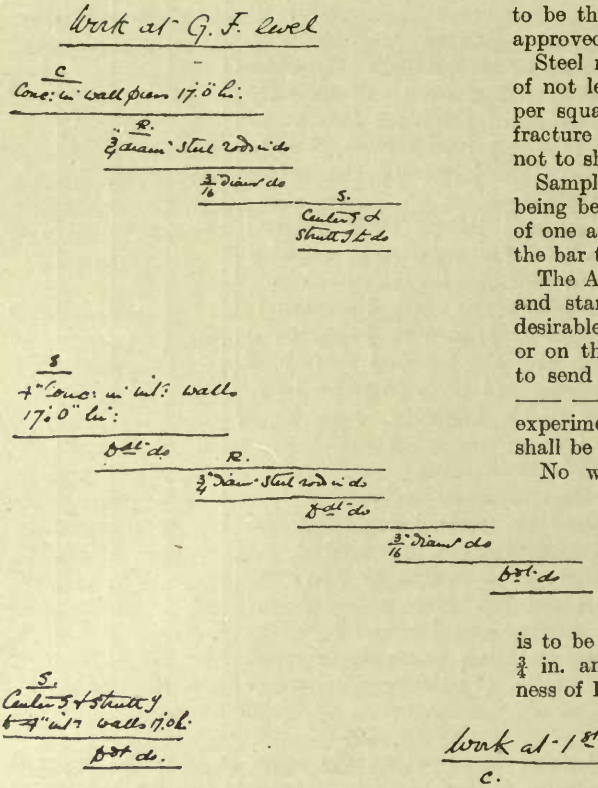
The setting out of a portion of an imaginary abstract on pp. 282 and 283 will illustrate what is meant.

BILLING

When one contractor is to carry out the whole of the work, including the reinforced concrete, all preliminary works, whether in connection with the general or the reinforced work, would find their way into the one preliminary bill. Many of the items, such, for example, as "water," would be covered by the usual clause, and would obviously apply to the whole of the work.

When the reinforced concrete work is to be made a separate contract, it must have its own separate bill of preliminary works, and it is often necessary to refer from one contract to the other in order to make it clear to each contractor exactly how much is expected of him and how much the other will have to do, as, for instance, in the provision of water, scaffolding, hoisting tackle, etc., previously referred to (*see* p. 271).

The Preamble.—Having written the "Preliminary Bill," the "Bill for Reinforced Concrete Construction" must in either case start with a "Preamble," most of which will be taken direct from the "specification." Judiciously worded clauses in the preamble will often save repetition of descriptions in the bill. The system of reinforcement to be adopted should be stated in the preamble, as the amount of labour in placing the steel varies with the different systems. Where it is found necessary in a few positions to adopt a special system of reinforcement the words "except where otherwise stated" should be added after the description of the system, and the special description of the exceptions given opposite the items. The nature of the clauses forming the preamble is indicated when treating of specifications, but clauses which relate solely to design will, of course, be quite superfluous. The following example will serve as a guide, but



it must be added to, and altered as necessary to meet the special requirements of each case :—

No. 2—Reinforced Concrete

The reinforced concrete construction will be on the —— system (except where otherwise stated) and the whole of the steel is to be obtained from——

The materials including the cement are to be tested as hereafter specified and the cost of all such tests whether accepted or not are to be borne by the Builder.

The cement to be portland from an approved manufacturer to be slow setting in accordance with the British Standard specification and capable of meeting the tests therein specified.

The sand to be river or pit sand coarse and gritty with grains of various sizes and perfectly clean.

The aggregate to be composed of clean gravel carefully selected and screened to pass a ¾-in. mesh.

The concrete to be composed as follows :—

Gravel	27 cub. ft.
Sand	13½ cub. ft.
Cement..	5½ cwt.

to be thoroughly mixed in a mechanical mixer approved by the Architect.

Steel must have an ultimate tensile strength of not less than 28 tons or more than 32 tons per square inch with a contraction of area at fracture of not less than 40 per cent. the fracture not to show more than 10 per cent. of granular.

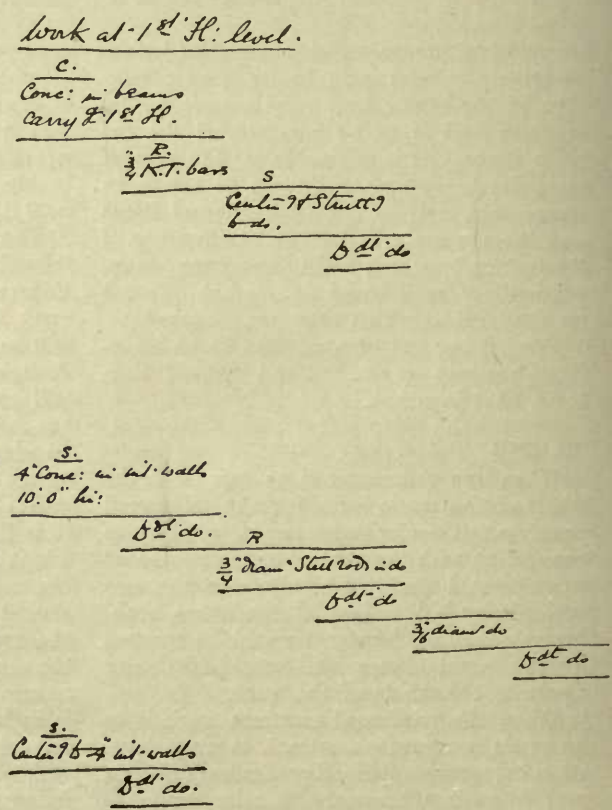
Samples of the steel bars must be able to stand being bent cold until the ends close over a bar of one and a half times the diameter of that of the bar to be tested.

The Architect or his representative will select and stamp such samples as he may consider desirable at any time either at the steel works or on the site and the Builder will be required to send such samples carriage paid to Messrs. —— —— —— or other approved testing and experimenting works and the reports obtained shall be accepted by the Builder.

No welding of the rods or bars will be permitted. They are to be bent cold.

The steel is to be brought upon the site as required and no steel is to be fixed in a dirty condition.

The steel reinforcement to slabs is to be protected by a minimum thickness of ¾ in. and that to beams by a minimum thickness of 1½ in. of concrete.



Sundries in Concrete

Runs — Numbers

4" Conc: Curbs 8" hi: on flats for Lantern Lights &c.

Lab: build^g in iron bolts wi: back plates incl: setting out & fixing bolts in position & all holes in centering.

Leaving holes 4" diam in 4" slab incl: temp^y wood drums for same

The workmanship throughout is to be of the very best description and to the entire satisfaction of the Architect who shall have full power to have any parts of the work which are in his opinion unsatisfactory re-executed to his satisfaction and the Builder is to bear the expense.

The centering to be true and rigid and of sufficient strength easily to carry the dead weight of the construction as a liquid without deflection; the posts or struts to be properly braced in both directions. All joints to be tight so as to prevent leakage. The centering to be so designed that the sides of the beams can be taken down first then the slab centering complete.

No centering is to be removed in less than four weeks' time after concreting or until the concrete has thoroughly set and has aged to give it sufficient strength to carry its own weight and in addition whatever live load is liable to come on the work during the course of construction. Beams are to remain supported at least two weeks after all other falsework has been removed. Columns shall not be given their full loading in less than five weeks after concreting. In no case shall any centering be removed without the consent of the Architect.

The Builder shall at his own expense test such parts of the work (not being more in the aggregate than 5 per cent. of the total area) as the Architect shall direct in the following manner: A dead load equal to the calculated super load plus 50 per cent. shall be evenly distributed over such parts of the floor etc. as are to be tested and the deflection shall in no case exceed $\frac{1}{600}$th part of the span of the slab or beam under test. The extra load of 50 per cent. shall be removed immediately after the reading and the calculated load shall be left for at least 12 hours in position when it shall be removed; fresh readings taken 24 hours later shall record no appreciable set.

No cracking or scaling of the concrete shall be developed under any test.

In the event of the aforesaid deflection being exceeded or an appreciable permanent set being found to remain or of the results of the test of any portion being otherwise unsatisfactory to the Architect that portion at the expiry of six

Sundries in Centering

Runs

Cutting & wasting to wall centering around openings

Do do to Slab Centering

Shutting to edge of conc: around openings in 4" slabs

Do in 6" do

Shutting to both sides of 4" Conc: Curbs 8" hi: on one side & 13" hi: the other

Ext: lab: to forming splayed int: angle 2" wide between wall & slab centering

Do do between beam & slab centering

Centering to edge of conc: around openings in 4" walls

Do: do: in 6" do

1½" × 1½" spl^d fillet fix^d on int: angle of beam centering.

weeks from the date of the first test shall be again tested in a similar manner. Should further deflection take place during the second test or further permanent set be found to exist or should the work in the opinion of the Architect still prove unsatisfactory then and in such case the Architect shall have full power to require the removal of the unsatisfactory portion and its reconstruction in a satisfactory manner at the expense of the Builder.

The test load shall not be applied to any part of the works until a period of two calendar months shall have elapsed since the deposit of the concrete in situ.

No cranes or other machinery shall be placed upon the reinforced concrete work and the extent to which the beams floors etc. may be loaded with materials during the progress of the work shall be subject to the approval of the Architect.

The price of the steel is to include for all labour in bending and fixing.

Other Clauses in the Preamble.—The preamble to a bill should include all clauses from the specification which may in any degree affect the prices of the measured items in the bill, as well as any general descriptions of material or labour which may save repetition in the descriptions of the measured items. It is not usual to include in the preamble any items requiring a price to be attached and for this reason in a bill of quantities for ordinary builder's work it has been the custom to place any clauses for testing work in situ (as, for example, in the case of the plumber's bill) at the end of the trade and to start the item with the words "Allow for testing, etc., etc."

The Testing Clauses.—The testing clauses in a reinforced concrete bill are not at present usually priced, but in a large job they must carry an appreciable money value which cannot reasonably be spread over the various items in the bill; the proper method of dealing with this item would be to put the clauses describing the method of testing in the preamble as in the foregoing example, and to insert as the last measured item in the bill the following: "Allow for testing the reinforced concrete construction as described." The contractor will then be at liberty to price this item or not, as he thinks fit. As competition gets keener, and with more detailed bills to help him in pricing each item as closely as possible, it may well be that he will embrace the opportunity of attaching a separate price to the testing.

Examples of Billing Measured Items.—It is always advisable in billing to adopt some definite order, but it is by no means essential to adopt that suggested in the section on "Abstracting." The descriptions, however, must be absolutely clear and definite so as to leave no doubt in the mind of the estimator as to what is meant, and they should also be as concise as is consistent with this rule. The example below illustrates the suggested system of billing the measured items following immediately after the preamble.

Then proceed to bill the next floor in a similar manner, and so on, floor after floor, finishing with the roof or flat. Having billed the whole of the principal items in this manner, proceed to bill the incidental and labour items under separate sub-headings of "Sundries in Reinforced Concrete," and "Sundries in Centering" (*see* p. 215).

These two examples illustrate the principles to be followed in billing, and from

Yds.				
200				Sup. 6" concrete in external walls of ground floor not exceeding 17'0" high above floor
390				" Plain centering & strutting to do (both sides measured)
Cwts.	qrs.	lbs.		
11.	3	7		Wt. In steel reinforcement to do.

Work on First Floor 15'0" above Ground Floor

Yds.				
100				Sup. 5" concrete in decking
				" Plain centering to do including strutting & bracing
Cwts.	qrs.	lbs.		
107.	0.	16		Wt. In steel reinforcement to do
Yds.				
3				Cube Concrete in beams
24				Sup. Centering to do including strutting & bracing
Cwts	qrs	lbs		
6.	3.	0		Wt. In steel reinforcement to do.
Yds.				
1				Cube Concrete in padstones to ends of beams
5				Sup. Centering & strutting to do.
Cwts.	qrs.	lbs		
	2.	7.		Wt. Steel reinforcement to do.
Yds.				
118	-	-		Sup 6" concrete in external walls not exceeding 10'0" high above floor
225				" Plain centering & strutting to do.
Cwts.	qrs	lbs		
6	3	7		Wt. In steel reinforcement to do

Sundries in Reinforced Concrete

Feet	Ins	
100	—	Run Groove 3/4" deep for tuck in of Asphalte including fillet on centering
40	—	" 4" Concrete Curbs 8" high on flats to receive lantern & lights
60	—	" Concrete in shaped core for moulded cornice 12" high & 12" projection
No. 80		Labour building in iron bolts with back plates including setting out & fixing bolts in position in centering and all holes in centering
" 104		Leaving holes 4" dia. in 4" slabs (same) including temporary wood drums for

Sundries in Centering

240	—	Run Extra labour to forming splayed internal angle 2" wide between beam and slab centering
60	—	" Cutting and waste to wall centering around openings
50		" Do. do to slab centering around do.

them the student should be able to bill and describe any other items, always bearing in mind that the object of a bill of quantities is to convey to the mind of the estimator in the most concise manner possible exactly what the builder has to do and how much of it.

A schedule of rates to be paid in day-work similar to that described on p. 271 should be given at the end of the bill.

PRICING

With regard to pricing the estimate, it is only necessary to examine a dozen tenders for the same work to see that pricing is not an exact science, and therefore any prices which may be given can only be approximate or average prices. The method of arriving at the total value of each item is by analysis, making due allowance for any special circumstances peculiar to each job. The total quantity of reinforced concrete will determine whether it will be more economical to mix the concrete by hand or to employ a mechanical mixer (if this has not been already settled by the architect).

The value of a cubic yard of concrete of any given composition will remain more or less constant, and may be readily ascertained, but the relative value of hoisting, depositing and tamping in various positions can only be assessed after considerable experience and close observation; and there are no reliable data at present available for general use.

In the case of a standard bar with fixed shear members, the relative value of fixing in different positions will vary comparatively little, but the value of hoisting will, of course, vary with the height.

The centering, if properly measured, can be priced with reasonable accuracy, and a little experience should enable the estimator to judge the relative proportion to be adopted for use and waste in different positions.

Prices for Concrete.—Concrete composed of $5\frac{1}{2}$ cwt. of cement, $13\frac{1}{2}$ cub. ft. of sand, and 27 cub. ft. of gravel, is equivalent to a proportion of 1 : 2 and 4 by measure; a cubic yard of this finished concrete requires nearly $1\frac{1}{2}$ cubic yards of dry materials, say 40 cub. ft., which, divided up into the given proportions (1 : 2 and 4), would give $\frac{40}{7}$ cub. ft. of cement, $\frac{2 \times 40}{7}$ cub. ft. of sand and $\frac{4 \times 40}{7}$ cub. ft. of gravel. $\frac{40}{7} = 5\frac{5}{7}$ cub. ft. of cement at 90 lb. per cub. ft. weighs 514 lb.; these data, together with the prime cost of the materials, including cartage to the site and the cost of mixing, will give the prime cost value of a cubic yard of concrete. The cost of water will be covered by the item for water in the preliminary bill if this item has been priced; if, however, the contractor has only attached a price to this item to cover fees and the cost of the temporary plumbing, the concrete

Feet	ins.	
240	—	Run $1\frac{1}{2}" \times 1\frac{1}{2}"$ splayed fillet and fixing in internal angle of beam centering to form splay on external angle of beam
90	—	" Shuttering to edges of openings in 4" slabs
120	—	" Centering around edges of openings in 6" walls
40	—	" Vertical centering to both sides of 4" concrete curb and edge of slab 8" high on one side & 13" on other
60	—	" Extra materials & labour over vertical centering for centering to shaped core as sketch 12" high & 12" projection

will have to bear its proportion of the water charges. In cases where the water is paid for by meter, about 30 gallons should be allowed per cubic yard of concrete. The cost of mixing will depend on whether it is mixed by hand or by machine. The cost of labour in hoisting, placing, and tamping will depend upon the height of the hoisting, and whether the work is in walls, slabs, sloping roofs, beams, etc., etc. To the cost of the concrete in situ so obtained must be added a percentage to cover builder's profit and establishment charges. The following example, to which average London prices have been attached, will explain the principle of analysis of a cubic yard of concrete :—

	s.	d.
Portland cement, 514 lb. at 30s. per cement ton delivered .	7	0
Sand, $11\frac{1}{7}$ cub. ft. at 6s. 6d. per cub. yd.	2	9
Gravel, $22\frac{2}{7}$ cub. ft. at 7s. per cub. yd.	5	11
Labourer measuring and mixing, 3 hrs. at 7d. . . .	1	9
Prime cost of a cub. yd. of concrete exclusive of hoisting and placing	17	5

To this price must be added the value of hoisting, placing, and tamping in various positions, and a percentage to cover profit and establishment charges. For example, a yard super of floor slab 6 in. thick will take $\frac{1}{6}$th of a cubic yard of concrete, and the analysis will be as follows :—

	s.	d.
$\frac{1}{6}$th of a cubic yard of concrete as above	2	11
Concretor placing and tamping, and Labourer hoisting not exceeding 20 ft., and attending upon concreter		4
	3	3
Profit and establishment charges 15 per cent.		6
	3	9

In a similar manner may be found the price of a floor slab of any other thickness or at any height, or the price of any item of concrete, whether billed in yards super or yards cube ; the prime cost of a yard cube, as given above, is the basis in every case.

The above analysis is for hand-mixed concrete. If a machine mixer is used, the cost of the motive power per hour, and the wages of the attendants must be ascertained, to which must be added a proportion of the cost of installing the plant, and a percentage on the capital cost of same to cover wear and tear ; from the cost of running the mixer thus found, and the amount of concrete which it is capable of turning out per hour, the cost of mixing per yard cube is readily ascertained.

Prices for Steel Reinforcement.—The cost of steel for the reinforcement, delivered on the site, will depend upon the bar used, and the distance for carriage and cartage ; to the cost of the steel as obtained from the makers, and carriage, must be added the cost of labour in hoisting and placing according to height and position ; the amount of such labour will vary with the system adopted, from the minimum amount in the case of bars with fixed shear members to the maximum in the case of ordinary rods where the shear members have to be bent to pattern and fixed in position. The Kahn trussed bar is an example of a bar with shear members in one piece with the bar.

The bar is rolled with side flanges which are machine cut ready to be turned up at an angle of 45° to form the shear members ; the price of the patent bar is about £14 per ton, or 14s. per cwt., at London station, to which must be added the cost of cartage to site, hoisting, fixing in position, and profit and establishment charges. The total cost of the bar, in London, fixed in position, averages for floor slabs and rectangular beams supporting same 15s. 3d. per cwt. ; for padstones at ends of beams, 15s. 6d. ; for ground floor walls 15s.

Prices for Centering.—The cost of materials for centering remaining constant, the value of centering in any given position will be governed by the proportion to be allowed for use and waste, the amount of labour involved in that particular position, and the quantity of bearers, struts, and braces per unit of area. Centering to floor slabs 12 ft. high will require, on an average, about 18 cub. ft. of timber for bearers, posts and braces, and the cost of materials for a square of centering will be as follows :

	s.	d.
1 square of 1 in. boarding at 13s. per square	13	0
18 cub. ft. of timber in bearers, posts, etc., at 1s. 7d. per cub. ft.	28	6
Cost per square	41	6

which is equivalent to 3s. 9d. per yard super.

It is usual to charge one-third the cost of materials for centering of this description for use and waste, on the assumption that they may be used three times before being discarded, each re-use entailing, of course, further cutting and waste to suit the several positions; on this assumption the cost of a yard super of centering to floor slabs 12 ft. high would be made up as follows:—

	s.	d.
⅓rd of 3s. 9d. for use and waste of materials	1	3
Cost of labour and nails erecting and removing at 7s. 6d. per square		8
	1	11
Add 15 per cent. for profit and establishment charges		3½
Cost of centering and strutting per yard super	2	2½

Where heights vary the timbering will not vary in the same ratio, for the reason that, while the joists and horizontal timbers remain the same, the posts and braces will vary both in length and scantling. The amount of timber making up the 18 cub. ft. per square of centering in the foregoing example may be split up nearly into 7 cub. ft. in horizontal members and short battens, and 11 cub. ft. in posts and braces, costing respectively 11s. 1d. and 17s. 5d. per square, which is equivalent to 1s. and 1s. 6d. per yard super, ⅓rd of which for use and waste amounts to 4d. and 6d. respectively. The latter prices may, within limits, be taken for all practical purposes to vary directly as the height, and so charged as ½d. per foot of height; any considerable increase in height will, however, necessitate an increase in scantlings, and will, therefore, upset this ratio. Beam centering or centering for any other purpose may be worked out in a similar manner; the relative amount of labour will vary in each case, so will the proportion to be adopted for use and waste, as previously explained, and the value of these these two factors can only be ascertained by experience. They can never be stated with mathematical accuracy, because they will be found to vary more or less on every job, and the amount of labour particularly will vary under different foremen. It is a good practice to take every opportunity of noting on each job for future reference the relative amount of waste and cost of labour in different positions.

It is a good exercise for the student to make drawings, however rough, of centering and strutting for different positions and heights, and to price them in detail in accordance with the foregoing principles.

Approximate or Average Prices.—As before stated, the prices given must be taken as approximate or average only. Prices of material and labour are continually changing and competitive prices vary still more, according to the state of the building trade. Builders, when trade is slack, are willing to cut their profit to almost vanishing point in order to keep their works going. From two recently priced bills, it is found that in one concrete varies from 21s. 9d. to 23s. 6d. per yard cube, and centering to floor slabs is priced at a uniform rate throughout of 2s. per yard, which is about 22s. 2¾d. per square; while in the other concrete is priced at a uniform rate of 28s. 6d. per yard cube, and centering to floor slabs at 17s. per square for a height of 12 ft., and 17s. 6d. for a height of 14 ft. It will be obvious from this one instance that any list of prices can only be taken as the merest guide. Moreover, the object here has been not to compile a price book, but to state the methods of measuring reinforced concrete work and to enunciate the principles on which the various prices are built up. In all new methods of construction it is essential that the quantity surveyor should keep himself acquainted with the broad principles governing their design and execution, in order that he may measure them intelligently and properly separate the different values. The estimator must keep himself posted in the current prices of materials, and by keen observation gradually build up data of the amount of labour entailed in the different items. He must also make himself acquainted with the cost of installation and removal of plant, percentage for wear and tear, etc., either for insertion in the preliminary bill or apportionment to the various items affected, as the case may be.

Arches and Bridges

DESIGN OF ARCHES AND BRIDGES

There is probably no branch of reinforced concrete work that shows the advantage of this material more fully than that of arches and bridges, as it lends itself readily to scientific design and architectural treatment, as will be seen upon referring to the many examples illustrated in this chapter. Some of the bridges that have been built, especially those of the open spandrel type, are really excellent in design and worthy of close study, being graceful and light, while at the same time satisfying the eye that they are sufficiently strong. Apart from being far more beautiful than steel bridges, the cost of maintenance with a concrete bridge is practically nil, and this is an important consideration when comparing the two materials.

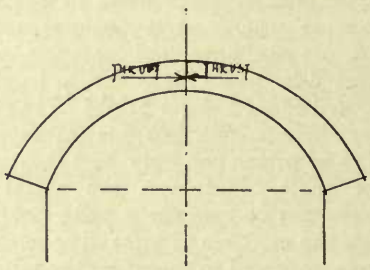


Fig. 479.—Horizontal Thrust at Crown of Arch

In order that the main principles which govern the design of arches may be understood, it will be advisable to give a few notes on the theory of arches in general, as the same principles will apply, regardless of the material employed. It is obvious that the arch is kept in equilibrium by the action of three forces, namely (1) the load or weight, which is equal to the weight of the arch itself and the superimposed load; (2) the reaction at the support or abutment; and (3) the thrust at the crown. In a symmetrical arch which is symmetrically loaded it is clear that the same line of thrust will occur on each side of the centre, and the thrust at the crown must therefore be horizontal and acting from the centre toward the abutments on either side; this is due to one half of the arch bearing upon the other half, as shown in Fig. 479, while the three forces acting on one half of the arch are shown in Fig. 480. The weight, however, will not be concentrated at one point in this manner, but will be distributed over the extrados of the arch, except in the case of live loads, which will be concentrated at every point in turn according to the progress of the load. It is usual when this is the case to calculate the arch to resist the load when it is at the most damaging point, and this will usually occur at one-fourth of the span. In the case of distributed loads, however, the horizontal thrust at the crown and the weight acting vertically at the various points will cause an oblique line of thrust according to their respective amounts. This line of thrust in arches built up of separate voussoirs should always pass through the middle third of the arch ring, but in the case of reinforced concrete this is not strictly necessary, as steel can be introduced to take tensional stress if the same should be found to be present upon calculation. The line of thrust should not, however, fall outside the arch ring.

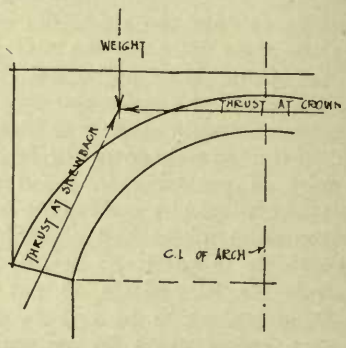


Fig. 480.—Three Forces acting on Half of Arch

This line of thrust is found graphically, as illustrated in Fig. 481. Here one half of the arch is shown, and it will be seen that the arch has been divided into equal portions and the forces acting downward

represent the load on each portion. The thrust at the crown is produced to meet W^1, which represents the weight on the first portion next the crown, and the two are combined to give the resultant R_1. This resultant is then again produced to meet the line of W^2, and these are combined to give R_2 and so on until the abutment is met, thus giving a line of thrust or resistance throughout the whole arch. If the arch were divided up into an infinite number of layers, then the line of thrust would become a curve, and in this particular instance a parabolic curve could be drawn without any other working, and this would be correct. There is one point, however, to be borne in mind, and that is the fact that there is no definite proof as to the point through which the horizontal thrust at the crown will pass, and the same remark applies to the thrust at the skewbacks, and although it is taken somewhere within the middle third of the ring, there is no law whereby the points can be determined with accuracy, and the line of thrust will be dependent on the points selected. The usual method in practice is to take the horizontal thrust at the crown as acting through the upper edge of the middle third of the ring, and at the skewback to take the lower edge of the middle third. There being three forces acting on the arch to produce equilibrium, it will be clear that the value of two of these will be dependent upon the other one, which will be the weight. The value of the maximum thrust will require to be found, and this can be done as illustrated in Fig. 482, where the arch is drawn to scale and the weight to be carried is also shown; if a live load exists this should be considered, and, if necessary, substituted by equivalent dead load, in order that the centre of gravity of the whole mass may be found. When the centre of gravity has been ascertained and

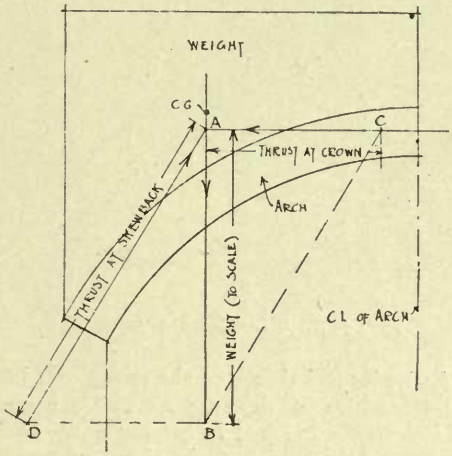


Fig. 482.—Forces Acting on Arch to Produce Equilibrium

put on the diagram as shown at C G, a vertical line is drawn through this point, indicating the downward tendency of the weight. The line of horizontal thrust is then drawn through the selected point within the middle third at the crown, and produced until it cuts the vertical line drawn through the centre of gravity as shown at A. The selected point within the middle third at the skewback is now joined to the point A, and the directions of the three forces is shown. The amount of the weight will be known, and consequently, if this is set

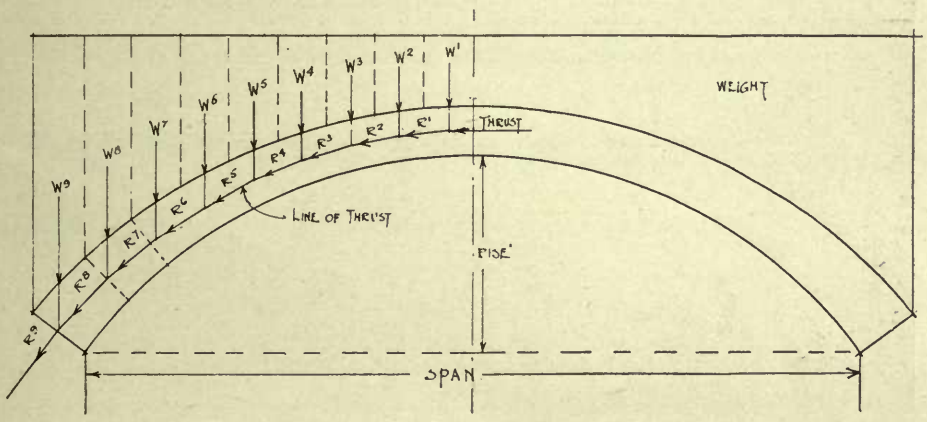


Fig. 481.—Finding Line of Thrust in Arch

downward to scale from the point A, giving the point B, and lines are drawn from the latter point parallel to the lines representing the direction of the other forces, as shown by the dotted lines, then the amount

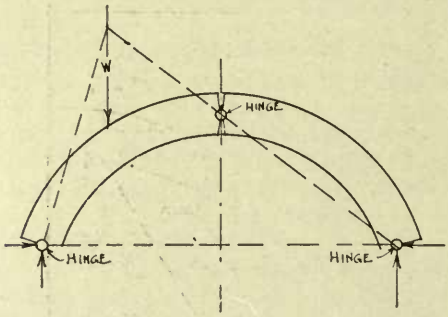


Fig. 483.—Three-hinged Arch

of the horizontal thrust at the crown will be represented by the distance A C to the same scale as that employed in setting down the weight, and the thrust at the skewbacks will be represented by the distance A D. If these three forces are set out separately they will give a closed triangle, which shows that equilibrium is produced, and the student will at once recognise an example of the

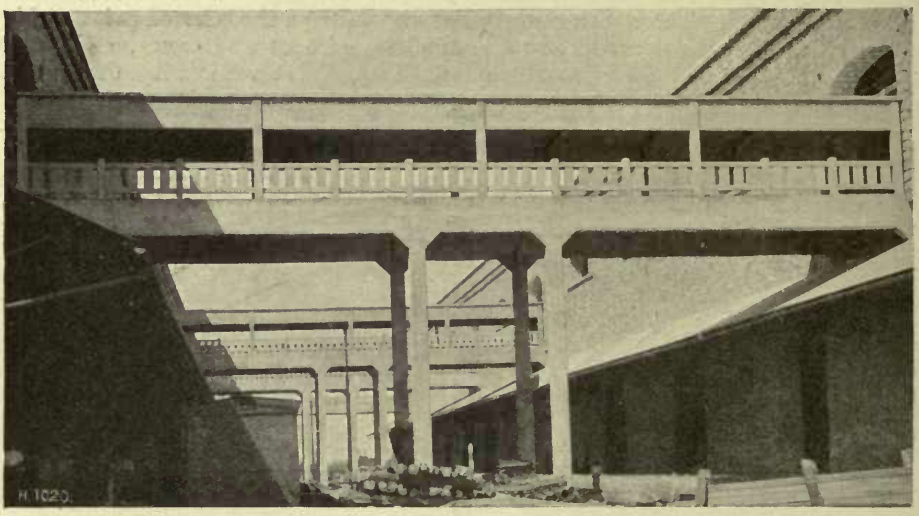


Fig. 485.—Foot-bridges at Railway Terminals, Vera Cruz

triangle of forces. The thrust at the skewback will always be in excess of the thrust at the crown, and for this reason the area of the concrete is often increased by making the arch of greater thickness at this point, as illustrated.

When the thrust has been determined the compression per square inch on the concrete can be found by taking the total sectional area of a width of the arch equal to 12 in. plus 14 times the area of the steel and divid-

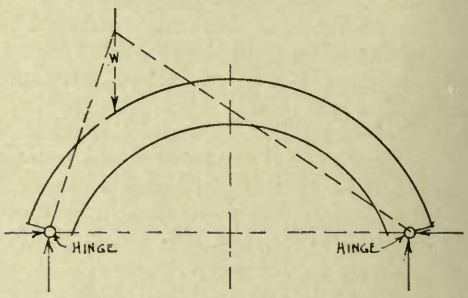


Fig. 484.—Two-hinged Arch

ing the same into the thrust as given for compression members. When the arch is unequally loaded in such manner that the line of thrust passes outside the middle third of the ring and produces tension, the moments must be calculated and the arch designed according to the formulæ given in an earlier chapter for double reinforced beams.

In addition to the type of arch described above, arches with hinges are sometimes employed, the introduction of hinges allowing the line of resistance to be absolutely determined, as it must pass through the definite points given by the hinges. Fig. 483 shows a three-hinged arch, and Fig. 484

a two-hinged arch. It will be clear that there can be no bending moment at these hinged joints, whatever the condition of loading may be. It is absolutely essential that the abutments are unyielding, and these must be properly designed to be safe against crushing or sliding. Provision to resist shearing and temperature stresses must also be made. Great care and experience are necessary in the designing of arches and bridges to produce satisfactory work, and the student is recommended to take a few simple cases, without introducing complications, and work these out and gradually lead up to the more advanced work, at the same time studying some of the many examples which have been carried out in practice.

EXAMPLES OF ARCHES AND BRIDGES

Foot-bridges at the railway terminals Vera Cruz, Mexico (*see* Fig. 486), are interesting constructions, one of them being no less than 993 ft. long, there being sufficient clearance for four tracks. Two expansion joints provide for variations in length. In constructing the railing, the top and bottom rails, with the uprights for one panel, were moulded on the ground and afterwards set in the forms for the railing posts, which were poured in place. The panels are 6 ft. 6 in. long, and can be lifted by three men. Much shorter bridges at the railway terminals are those between the Customs warehouses (*see* Fig. 485). These were designed for a loading of 200 lb. per square foot, and also for a track loading of push cars carrying 4-ton loads.

A foot-bridge over the Weisseritz, at Cotta-Dresden, is shown by Fig. 487. It is of

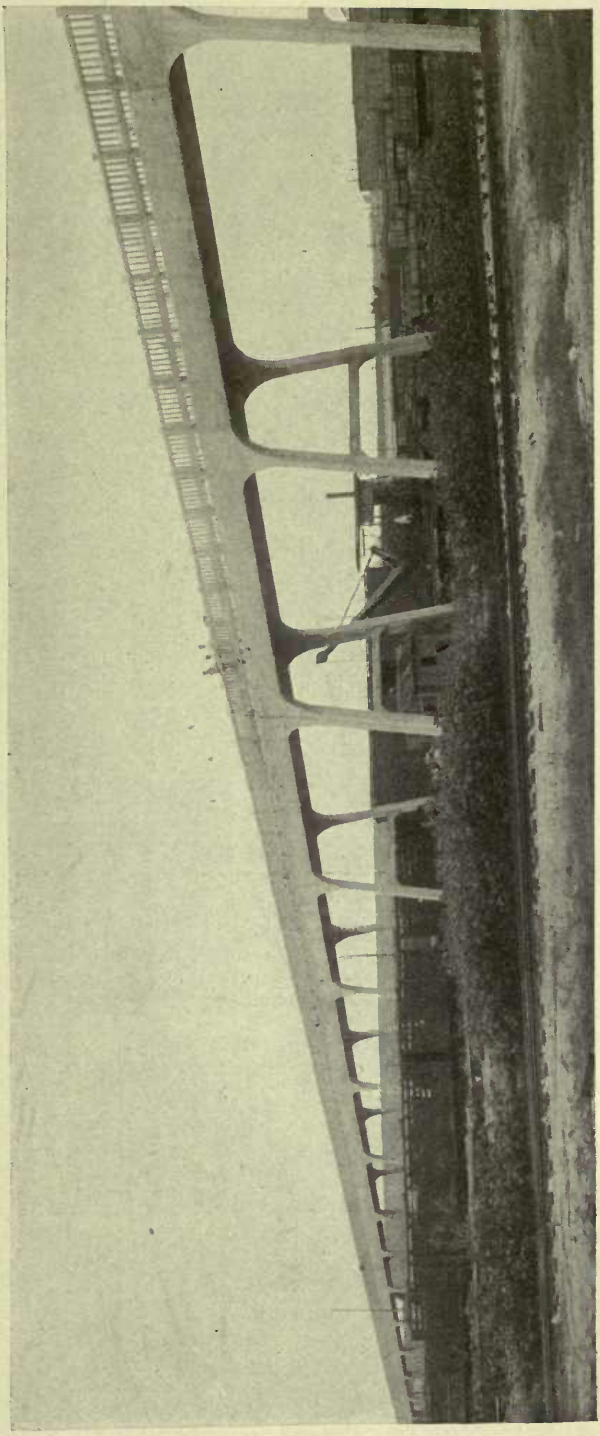


Fig. 486.—Foot-bridge at Railway Terminals, Vera Cruz

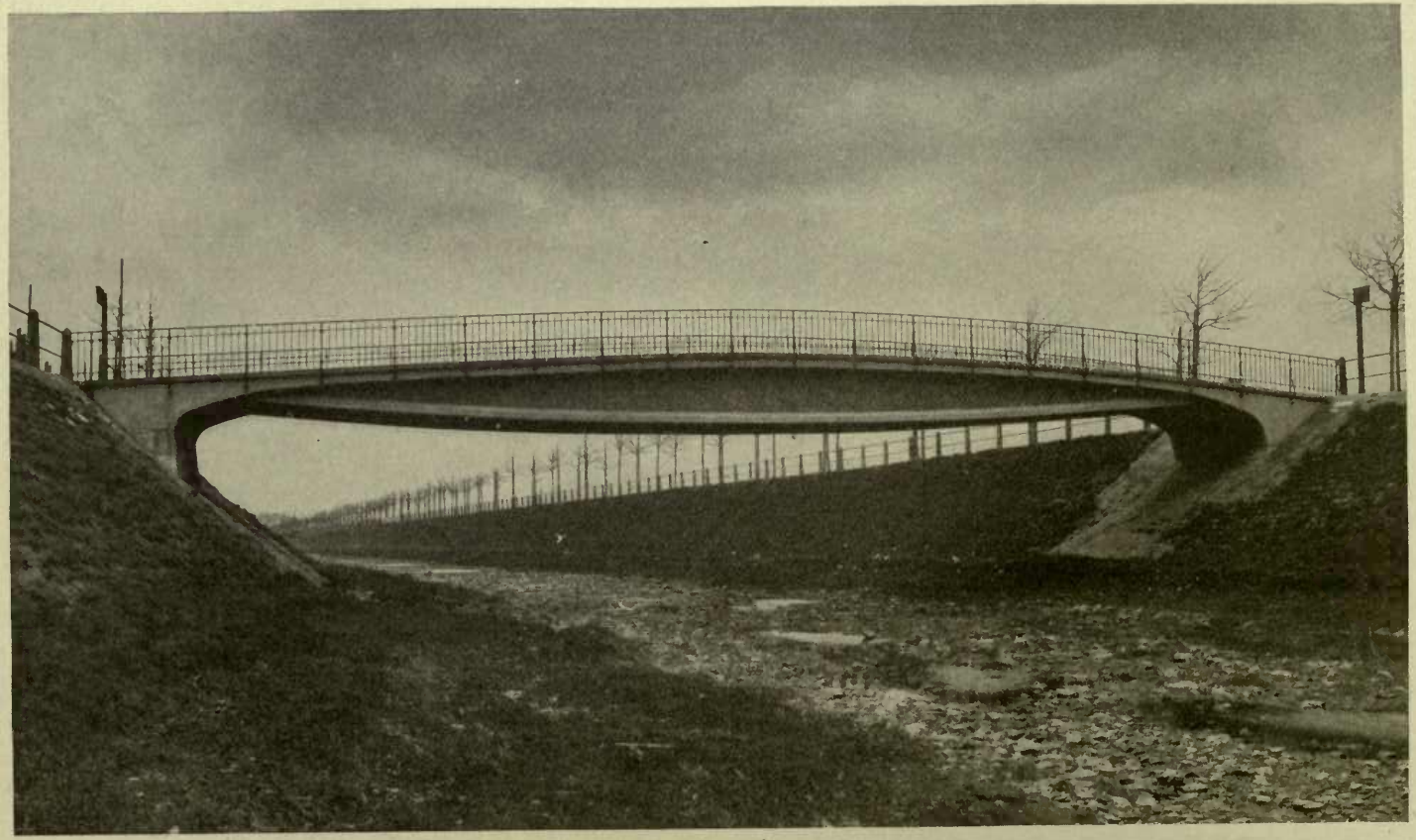


Fig. 487.—Foot-bridge over Weisseritz, at Cotta-Dresden

85-ft. span, and was erected on the Möller system of horizontal girder bridges, by Messrs. Rud-Wolle, of Leipzig.

Fig. 488.—Moortown Bridge, Wimborne, Dorset

The bridge at Wimborne, Dorset, shown by Fig. 488, is one of two bridges in reinforced concrete erected at Wimborne, one on the Moortown Road and the other on the Ringwood Road. The bridges were designed and executed by the Cubitt Concrete Construction Co., and the test load was carried out six months after completion, the recorded deflection amounting to only a little more than $\frac{1}{16}$ in.

A bridge in Cowley County, Kansas, consisting of a small steel span with 28 ft. of wood trestle approach, was converted into a reinforced concrete structure (*see* Figs. 489 to 491). The existing structure was dismantled, the abutments and piers repaired, the steel work cleaned and replaced, the additional reinforcements put in position and wired to keep their place. Timber

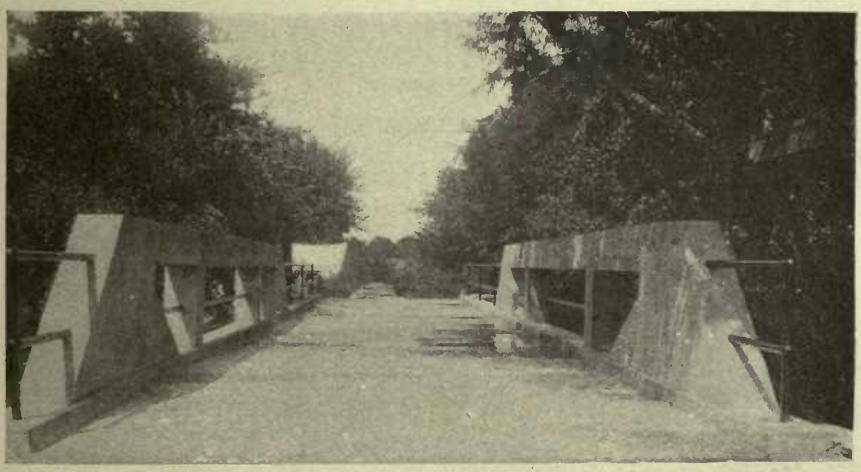


Fig. 489.—Decking and Parapets of Bridge in Cowley County, Kansas

Fig. 490.—Underneath View of Main Floor Span of Bridge in Cowley County, Kansas

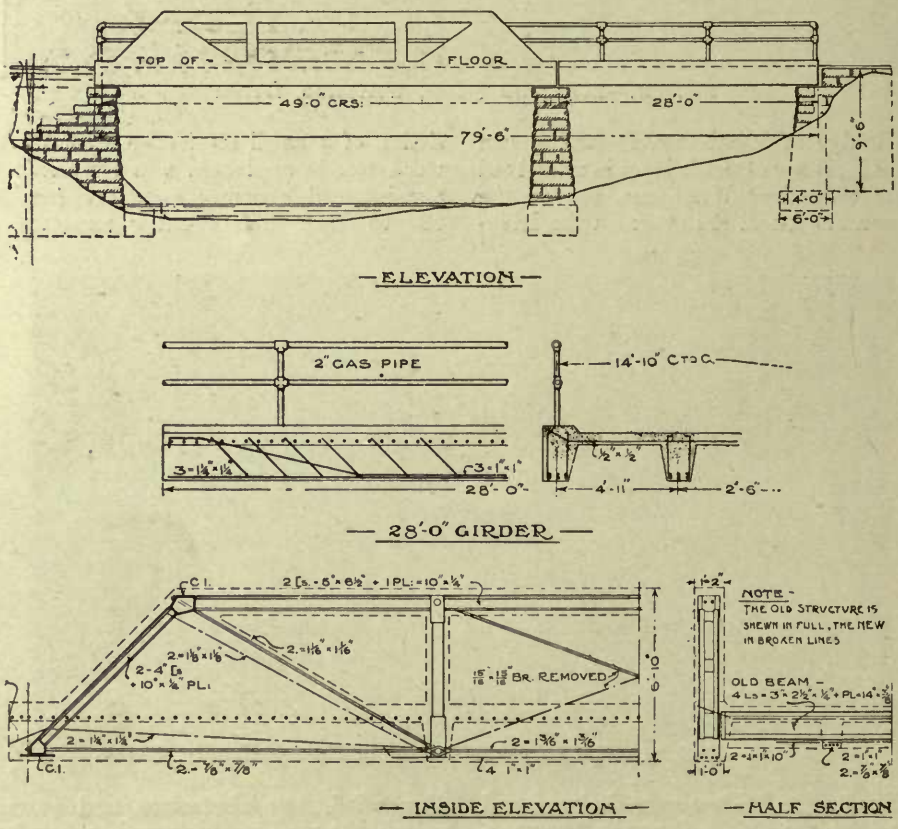


Fig. 491.—Elevation and Details of Bridge in Cowley County, Kansas

for the forms was 1-in. dressed boards for the side pieces, the braces to prevent bulging being 2 in. by 4 in. and 2 in. by 6 in. The span is 50 ft., the approach 28 ft. 9 in., and the road bed is 14 ft. wide, and it was estimated that the cost of the work was two-thirds that for a new concrete structure.

A bridge over the Merrimac River, New Hampshire, U.S.A. (Fig. 492), is an example of a bridge built under extremely difficult conditions, inasmuch as it replaced an old iron bridge and was built underneath it, so as not to stop the traffic. It was designed by W. M. Denman, of Springfield, Mass. The grade of the roadway of the new bridge was required to be 4 ft. higher than the road level of the old iron bridge. According to *Engineering News*, the old iron spans were 150 ft., 168 ft., and 170 ft. long, with an 18-ft. roadway and one footpath. The portal struts were very light, being only 18 in. deep, made of the lightest angles and lacing bars. In order to build the new bridge under it, it was necessary to raise the old spans, and the light portal struts made this a very difficult undertaking. The middle span was raised 7 ft. above its original grade, and the shore ends were put up 5 ft. The struts, however, began to buckle, and the bearings of the end-posts displayed a tendency to work out of line before the spans were half-way up. The light struts were stiffened by embedding them in concrete. Forms were built about them and concrete poured, while the traffic passed continuously over the structure, causing heavy vibration of all members. Nevertheless, the concrete set perfectly. One abutment for the new bridge rested on rock just within the old abutment, which is used as backing to resist the thrust. It was found necessary to underpin this abutment in sections with concrete. Two of the piers also rest on rock at 4 ft. to 6 ft. below low water. The two other piers are on piles in 10 ft. to 15 ft. of water. The piers of the new bridge were so spaced as to permit of their construction without interfering with the old piers. The arches of the new spans necessarily passed through the spaces between the tops of the old piers and the old truss bridge. This was done by boxing in

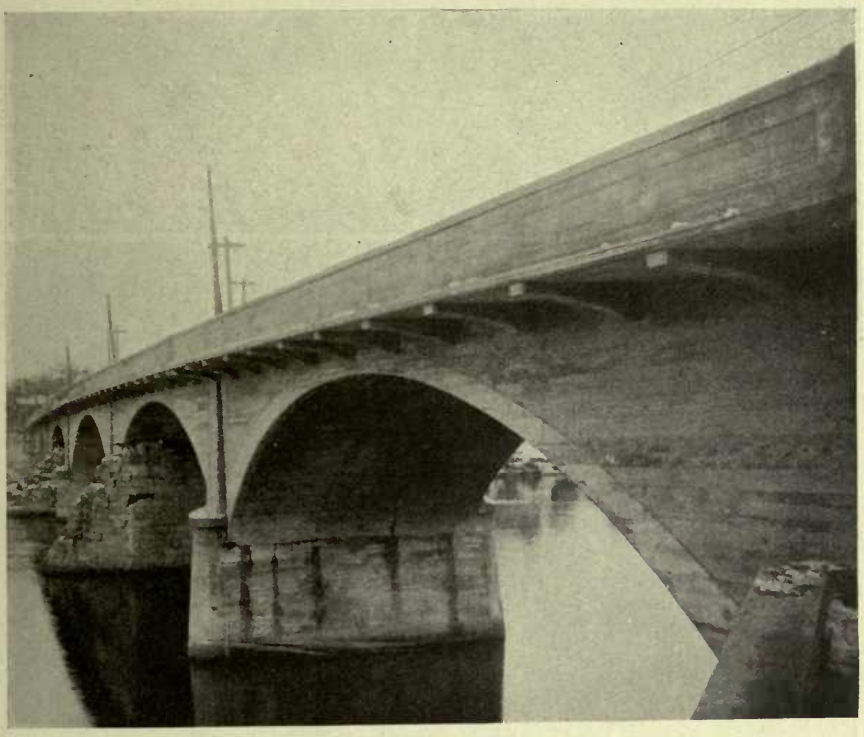


Fig. 492.—Bridge over Merrimac River, New Hampshire

Fig. 493.—Foot-bridge at Mizen Head

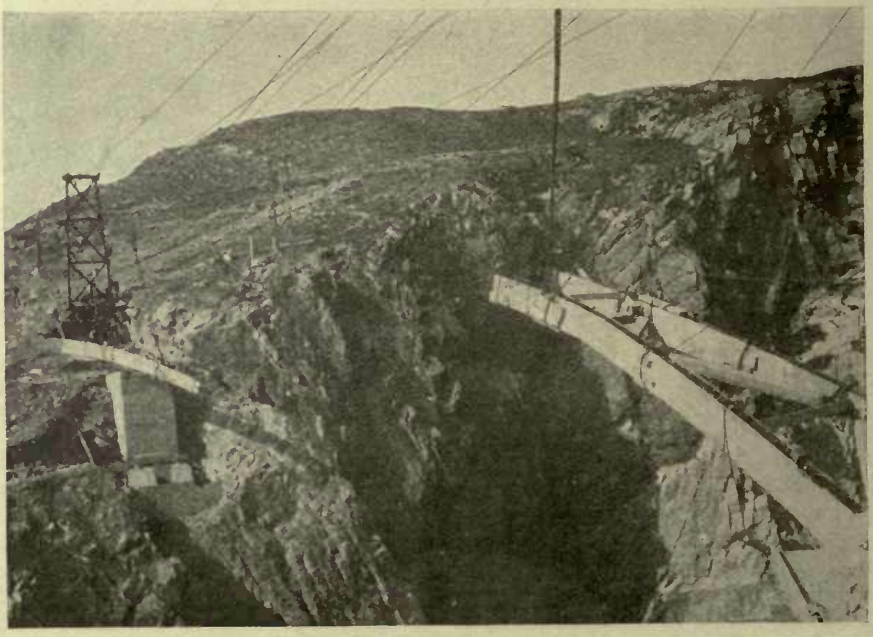


Fig. 494.—Erecting the Ribs for Foot-bridge at Mizen Head

the 4 ft. by 4 ft. columns of blocking under each end-post and concreting the arch ring around the box, diverting the steel reinforcement at that point. After the arches had hardened, the fill was made and the old bridge let down on the new, so that it could be dismantled and removed. The columns of blocking were taken out and the holes plugged with concrete. The new bridge has spans of 88 ft., 94 ft., and 100 ft. The roadway is 30 ft. wide in the clear, and the footpath is 6 ft. wide.

A one-span foot-bridge at Mizen Head, Ireland, giving access to Cloghan Island, is shown by Fig. 493; it was designed by M. Noel Ridley, the contractors being Alfred Thorne and Sons. The span is 172 ft., and the height above the sea 150 ft. The bridge is of the through-arch type, the ribs being of rectangular section, 3 ft. by 1 ft. at the springing and 1 ft. 6 in. by 1 ft. at the crown. The following are the chief dimensions: Ribs, 5 ft. 6 in. apart, centre to centre, where parallel in the centre portion, and 15 ft. apart at the abutments. Footway, 4 ft. 6 in. wide between horizontal beams (15 in. by 9 in.), and 5 in. thick at centre. Trestles and wind bracing, 9 in. by 9 in. Hangers, 9 in. by 6 in. Handrails, 4 in. octagonal, with posts 9 in. by 6 in. and 6 in. by 6 in. The work was done without centering, the ribs being made on shore and built

Fig. 495.—Suspended Centering for Bridge

Fig. 496.—Bridge Crossing the Bremba, Italy

Fig. 497.—Bridge at Kiel Dock Works

out from the abutments. The various parts were placed in position by means of a wire ropeway (*see* Fig. 494.) Only the floor and hangers were made in situ. Under a distributed test load of 150 lb. per square foot, the maximum deflection at the centre was about $\frac{1}{5}$ in.

Reinforced concrete is a popular material for bridge construction in Italy. Occasionally the designs are striking and even daring, an instance being the bridge that crosses the Bremba at the Gorge of Sidina (*see* Figs. 495 and 496). The first photograph gives some idea of the exceptional difficulties of the construction and shows the suspension of the centering from the old brick bridge. The arch, which has a span of 37·5 metres, is shown complete in the second view. The bridge was constructed by F. Leonardi, of Milan.

A triple-hinged bridge, giving distinct architectural effect, and built by Weirich and Reinken, of Kiel, is shown by Fig. 497. Its span is 27 metres, the rise being 5 metres. The parapet and roof are of wood, and the steps are of plain concrete formed direct into the arch.

A bridge over the San Luis Rey River at Oceanside, California, is shown by Fig. 498. It is a 700-ft. bridge of three arches, and its members were built on the ground, left there to harden, and then hoisted into position, the object being to obviate the excessive cost of false-work. According to *Concrete*, the arch ribs were poured in two sections, and the spandrel posts cast separately. After these were put in place on piers and abutments the forms were bolted in place for the deck of the bridge. The bridge was designed on the W. M. Thomas system (patented), the expected load being 250 lb. per square foot. It consists of six spans, four of which are 103 ft. long, the two end spans being 107 ft. The bridge is 19·4 ft. in width over all, with 18-ft. roadway.

The foundation for the piers and abutments consists of thirty piles to each pier and abutment, the piles being cut off 2 ft. below mean low tide, the foundations thus extending down about $3\frac{1}{2}$ ft. below the river bed. Each pile was calculated to sustain a 17-ton load. The foundations of piers are 12 ft. wide and 26 ft. long, and there is no reinforcement in the piers. The arch ribs or beams were made on the ground under the respective positions which they were later to occupy in the completed structure; they are 11 ft. on centres, and are reinforced with a steel framework consisting of twelve $\frac{3}{4}$-in. round rods and $\frac{1}{4}$-in. by $1\frac{1}{4}$-in. flats, the reinforcements being connected at the crown end of the beam to cylindrical plates of steel having a ball joint mated into a cup in the opposite beam. The semi-spherical protuberances at the reverse end of the beam are $\frac{1}{2}$-in. semi-circular plates, forming portions of the hinges at skew back on the piers. The hinge shoe on piers consists of 1-in. bolts, the rods extending deep into the concrete. The spandrel posts, also made separately on the ground, were poured in 12-in. by 16-in. sections. The longer posts are reinforced with four $\frac{5}{8}$-in. round rods, and the shorter posts have four $\frac{1}{2}$-in. rods. These rods were left protruding from the sides, top, and bottom of the posts. The rods extending from the top form a tie to connect the spandrel arches with the spandrel posts; the rods protruding from the bottom have a thread, and they extend through the rack beam and bolt the spandrel post to the beam; the rods extending through the sides of the beam are embedded in diaphragm cross braces, and hold the posts to a rigid position. Holes were left in the spandrel posts and the wooden centering for spandrel arches to provide means for firmly bolting the floor supports to the spandrel posts; the monolithic concrete deck was then poured in the usual manner. The floor was reinforced with $\frac{5}{8}$-in. round rods running transversely across the bridge, and the outside cantilever portion of the floor has rods running longitudinally 6 in. on centres.

The Scenery Hill Viaduct, Philadelphia (Fig. 499), includes an arch having a span of 33 metres over a railway line, and carries a roadway which is 32 metres above the rails. The construction is on the Hennebique system, with plain bars, to which are hooked stirrups acting as a secondary reinforcement.

The Meadow Street bridge, Pittsburg (Figs. 500 and 501), is 454 ft. long, having a 30-ft. roadway and two 10-ft. footpaths, with a maximum clear height at the centre of the arch of 78 ft. The main arch span is 209 ft. long, with a rise of 46·14 ft., and consists of three arch ribs, the two outside ribs varying from 5 ft. deep at the crown to 6 ft. 2 in. at the springing line. The main arch ribs are connected at the panel points

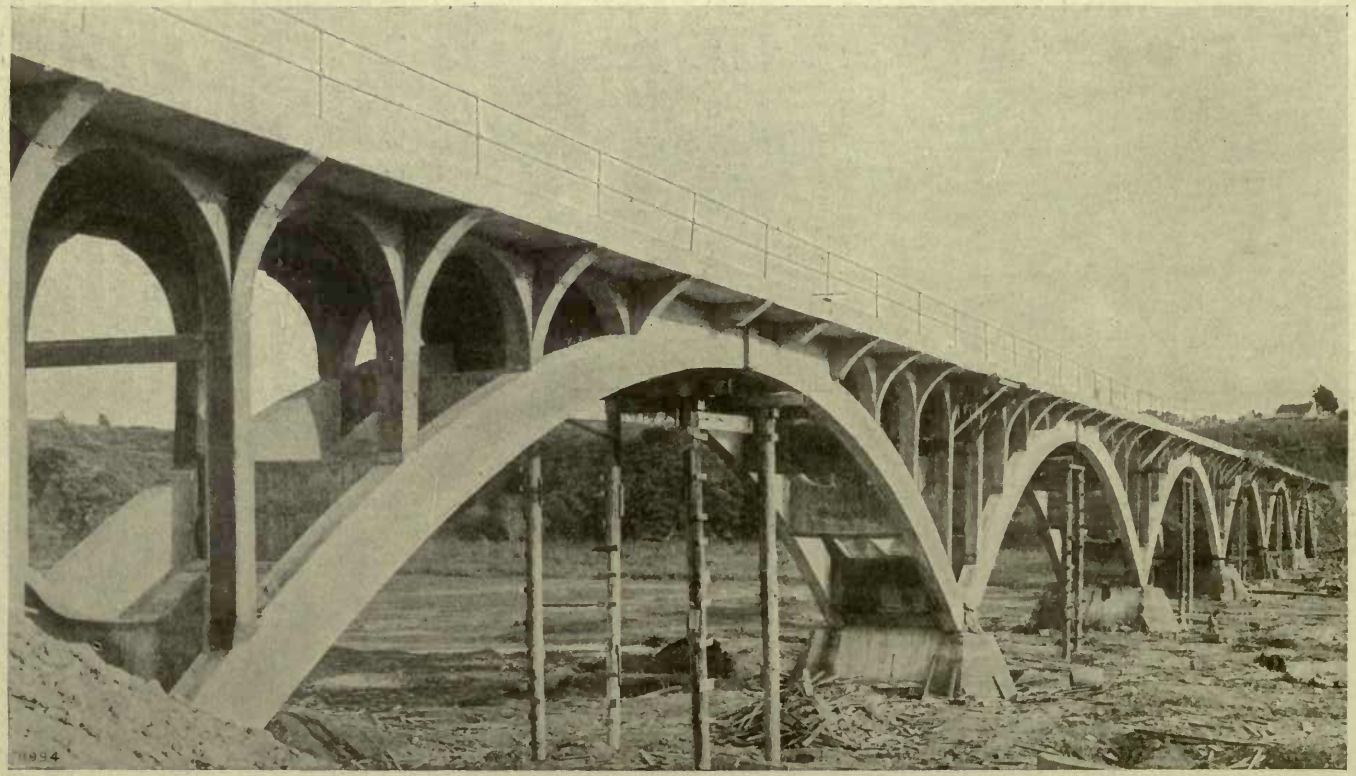


Fig. 498.—Oceanside Bridge over San Luis Rey River, California

by twenty-six horizontal struts, each 12 in. wide, which provide additional lateral strength. The main arch ribs spring from two main piers of cellular construction 37 ft. 9 in. long and 14 ft. in width. Between the main piers and the abutments are three viaduct approach arches, each having a span of 21 ft. The front and side walls of the east abutment are 13 in. thick, and are designed for vertical pressure and also to act as a beam if necessary, the tops of the walls being covered and tied together with a reinforced concrete slab, varying in thickness from 8½ in. at the centre line of bridge to 12¼ in. at the curb lines, similar to the type used on the main spans, and extending 28 ft. back from the face of the abutment. Beyond this point the side walls are designed as ordinary retaining walls with spread footings and to act as cantilevers. The side walls on the west abutment are 37 ft. long and designed as ordinary retaining walls, the same as the portion of the side walls of the east abutment which has been already mentioned.

The roadway of the bridge, which is of reinforced concrete slab and floor-beam construction, is supported on spandrel girders supported by columns 24 in. by 24 in., spaced 15 ft. centre to centre, resting upon the main arch ribs. The roadway on the approach spans is supported in a similar manner, except the span is 21 ft., and the columns, which are 24 in. by 36 in., extend to the rock below, the greatest length of the column of the approach spans being 39 ft., and on the main span 29 ft. in length.

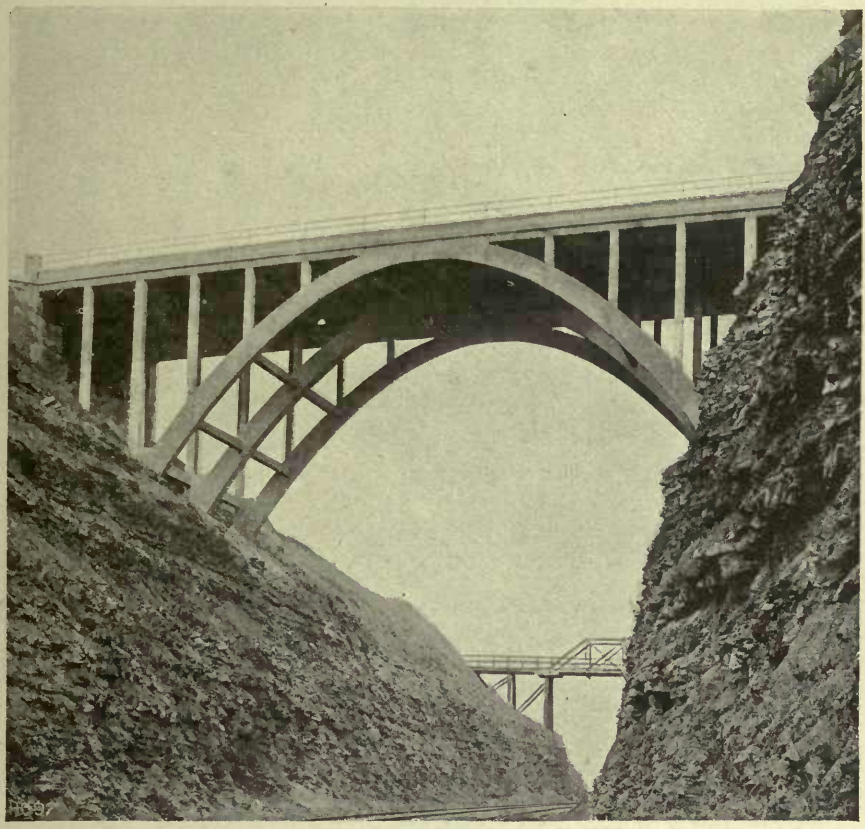


Fig. 499.—Scenery Hill Viaduct, Philadelphia

The footpaths, which are reinforced, have a uniform thickness of 6 in., with a slope of ¼ in. to the foot to provide drainage, are 10 ft. in width, and are supported on beams acting as cantilevers spaced 7 ft. 6 in. on the main span and 7 ft. on the approach spans.

The reinforcement for the main arch

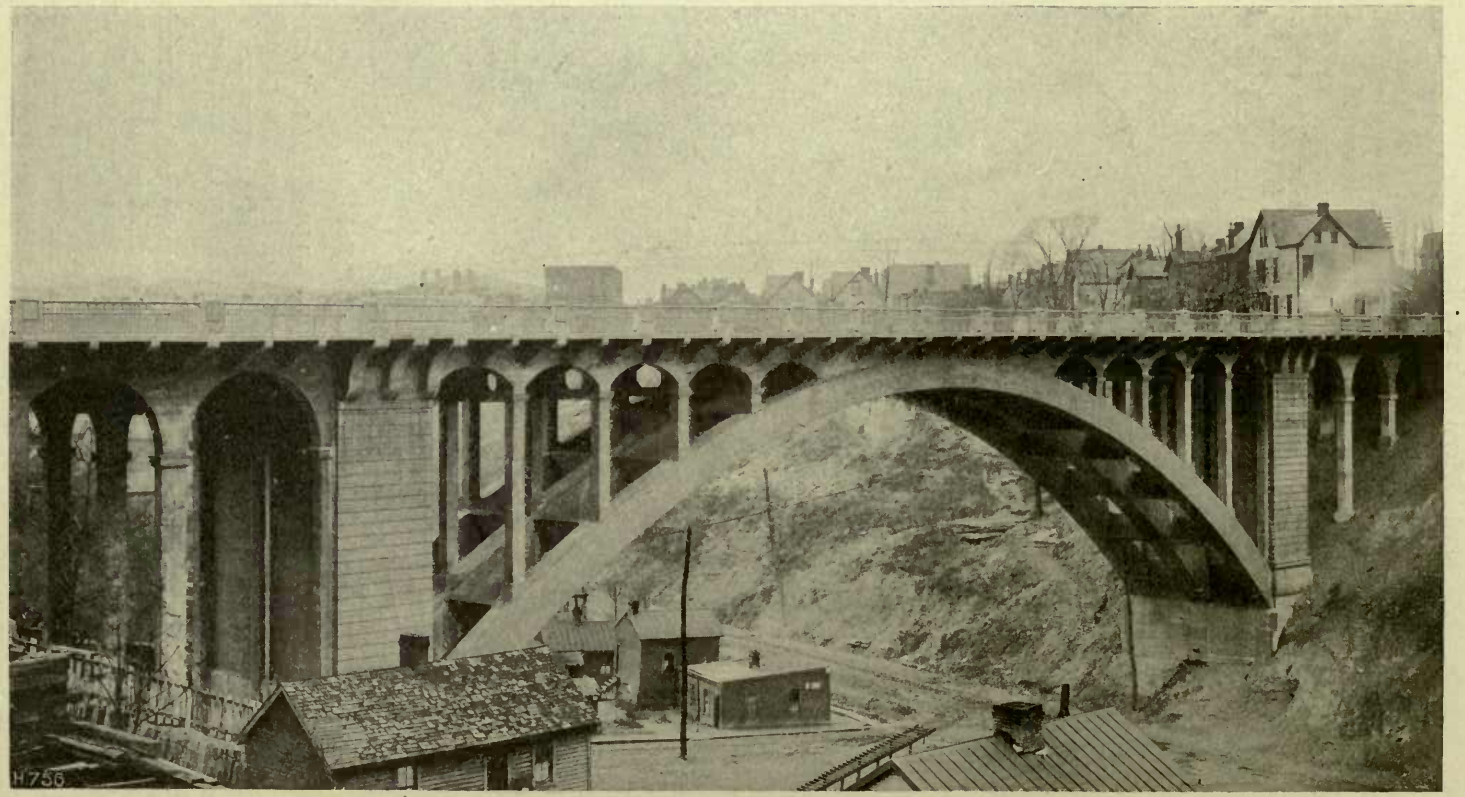


Fig. 500.—Meadow Street Bridge, Pittsburg

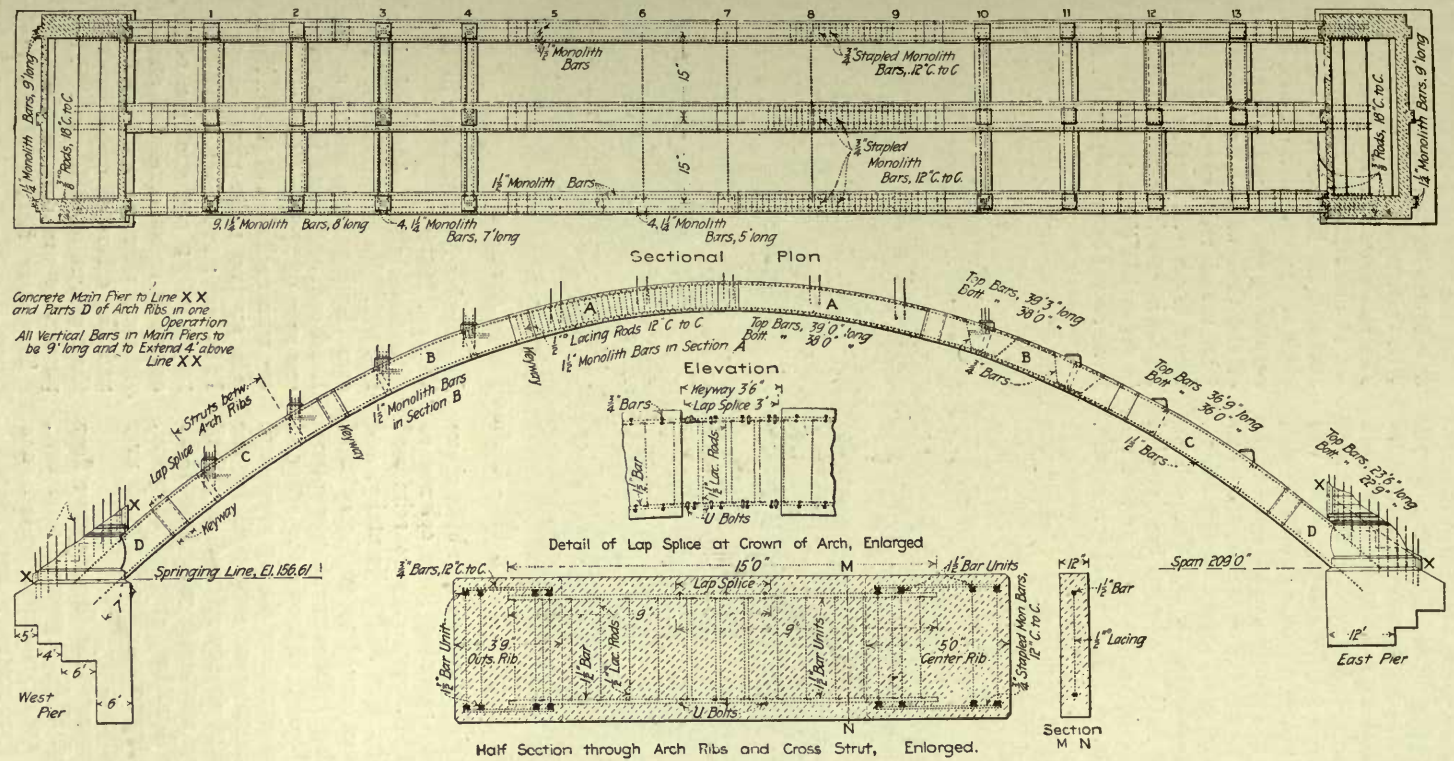


Fig. 501.—Sectional Plan, Elevation and Details of Meadow Street Bridge, Pittsburg

ribs consist of eight $1\frac{1}{2}$-in. grooved bars, arranged in four pairs, each pair connected with $\frac{1}{2}$-in. lacing bars, clamped into the grooves on the $1\frac{1}{2}$-in. bars, and spaced about 1 ft. apart, making a latticed girder effect, placed vertically and curved to fit the arch rib. The strut reinforcement, which was made in two pieces of latticed girder frames, similar to that of the ribs, was stapled to the longitudinal bars in the main ribs, the struts being placed at panel points on the bridge, the main bars being $1\frac{1}{2}$ in. The column reinforcement is $1\frac{1}{4}$-in. vertical bars, stapled at 9-in. intervals, and clamped around the hooped bands, which were $1\frac{1}{4}$-in. rounds. The spandrel girders, which were built above each arch rib, spanning from column to column on 15-ft. centres, were reinforced with five $1\frac{1}{2}$-in. bars laid horizontally, and two $1\frac{1}{2}$-in. bars curved to fit the soffit of the arch. Top reinforcement was placed over the columns, although the spandrel girders were designed as simply supported. The floor beams were supported by the spandrel girders already referred to, and all of them were reinforced with four $1\frac{1}{4}$-in. bars and one 1-in. bar, grooved as before, negative reinforcement being placed over the centre column.

The reinforcement in the roadway floor slabs consisted of $\frac{3}{4}$-in. grooved steel bars, spaced 6 in. apart, centre to centre, on a span of 7 ft. 6 in. centre to centre, between the floor beams in a slab of a minimum thickness of $8\frac{1}{2}$ in. The reinforcement for the footpath slab is $\frac{5}{8}$-in. grooved bars, placed 6 in., centre to centre, in a 6-in. slab of 7 ft. 6 in. span, between footpath brackets. The reinforcement for the floor system was set in place and wired to the forms before concreting. Round bars were used in the piers at the end of the main span near the surface of the concrete, to prevent cracking of the thin walls. The reinforcement was $\frac{3}{8}$-in. round rods, spaced 24 in. apart and wired in place. Further particulars of this highly interesting construction will be found in an article (from which the above details have been abstracted) by N. S. Sprague in *Engineering News* and also in *Concrete* (Vol. VI., pp. 203–209).

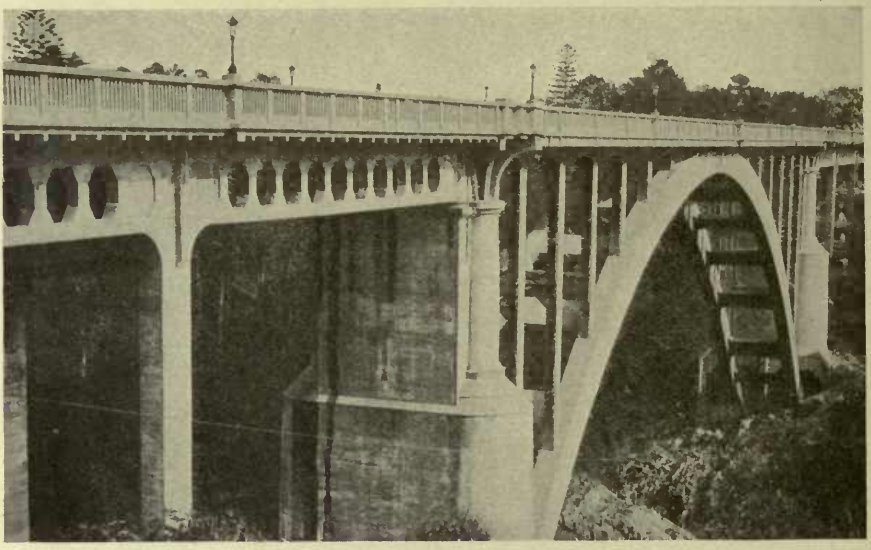


Fig. 502.—Grafton Bridge, Auckland

Grafton bridge, Auckland, is shown by Fig. 502. Its total length is 960 ft., a short span having been added at the Grafton Road end, and there is one span of 35 ft.; one span of 37·5 ft.; four spans of 75·5 ft.; one main arch span of 320 ft., flanked on each side by two main piers occupying at top 18 ft. each; two spans of 78·5 ft. and 80·5 ft. respectively; and one span of 42 ft. All approach spans consist of straight girders, the three shorter ones being of solid design and the others of open panel. There are three girders to each span. The following particulars are due to *Concrete:*

The main feature of the bridge is, of course, the 320-ft. 3-hinge arch (with a rise of 84 ft. from centre of abutment hinges

to centre of middle hinge), which consists of two parallel ribs ranging in size from 6 ft. by 4 ft. 2 in. at the abutments to 9 ft. 9 in. by 4 ft. 1 in. (in the girder) to 5 ft. by 4 ft. at the centre hinge, connected by wide ties of reinforced concrete 8 ft. 4 in. by 20 in., haunched 18 in. each way into the rib and spaced at 21-ft. horizontal centres. The end hinges rest on solid abutments or skewbacks of concrete, with a small amount of reinforcement, there being 720 cubic yards of concrete in the main abutments. These abutments rest upon a foundation consisting of a sort of sandstone, known locally as "papa" rock, and are quite independent of the large piers previously referred to, so that the movement in one would not necessarily affect the other. The hinges themselves, which consist of cast steel, rest on grills made up of steel H-girders and plates, the centre hinge being treated in like manner, the grills being built into, and surrounded by, concrete. The superstructure consists of seven piers of T-section, the T being 3 ft. by 6 in., and the remainder 2 ft. 6 in. by 10 in. spaced 18 ft. apart in the clear. At the abutment the decking is carried by the cantilever, which projects 7 ft. from the end pier, a similar cantilever being constructed from the main pier over the abutment to meet the one referred to. The three highest piers on each half are braced by horizontal transverse beams of T section, spaced at intervals of approximately 30 ft. On each pier rest transverse girders, 3 ft. 7 in. by 10 in., with haunches at each bearing; the actual deck system on the main span consists of five secondary beams, each 1 ft. 8 in. by 10 in., running parallel with the direction of the bridge, the deck itself being of reinforced concrete fully 6 in. thick. The kerbing is situated over the outer beam in each case, the footpath being carried by a cantilever consisting of a deep haunch occurring at each transverse girder and by a decking 5½ in. thick in between. A further secondary beam, 12 in. deep and 6 in. wide, connects the haunch beams under the parapet.

The open panel girders are 9 ft. deep by 15 in. wide, there being three girders in the width of the bridge. These girders are supported by piers. The piers are formed by two solid piers, 5 ft. by 4 ft., slightly reinforced with steel and connected at top by a deep girder or curved beam to carry the middle girder. The short spans, as before described, are ordinary solid girders, the two at the Symonds Street end being 3 ft. 9 in. by 15 in., and that at the Grafton Road end being 3 ft. 9 in. by 15 in.

The deck system in the case of the straight girder spans is the same in all instances, and consists of transverse secondary beams 2 ft. 6 in. deep at the middle girder, diminishing to 1 ft. 6 in. depth at the outer girder and of a uniform width of 6 in., and continued beyond the outer girder in the form of a cantilever to support the footpath and parapet. The decking on the carriage way is 6 in. thick, that of the footpaths being 5½ in., including a 1½-in. layer of fine concrete. The parapets are 4 ft. 9 in. in height, and constructed throughout of reinforced concrete. In the top member of the parapet is fixed black iron tubing to receive the cables for the electric lighting of the bridge. The construction of the bridge occupied two and a half years, and was carried out by the Ferro-Concrete Company, whose Chief Engineer, R. F. Moore, was responsible for the design, most of the calculations in connection with the bridge having been made by C. Rosegger. Under a test load, equal to 113½ lb. per square foot, the total amount of deflection registered ⅛ in., but from various causes too much notice must not be taken of this figure. In a later test, consisting of passing over the bridge simultaneously two steam rollers, weighing together 32 tons, the maximum deflection registered on the main span was $\frac{1}{12}$ in., and on the girder span $\frac{1}{15}$ in.

A bridge over the River Werra, in Thuringia, Germany (Figs. 503 to 505), was built by Christiani and Nielson. In design it consists of a rigid arch, the two-arched beams being united by transverse struts, and carrying the reinforced concrete roadway by means of steel ties. To prevent injury from variations of temperature, two end sections are rigidly connected with the arched beams, while the middle section hangs freely, engaging with the end ones by means of indentations. The clear span is 53 m. The decking is 12 cm. to 14 cm. (5½ in. thick), and is carried by four longitudinal beams, which in their turn rest on transverse supports at intervals of 4·15 m. (13 ft. 7 in.). These supports are 80 cm. (2 ft. 7 in). wide in the middle, tapering to 50 cm. (1 ft. 7½ in.) at the ends. The rails are continuous, and are carried on sleepers, which are readily removed sideways if required. The

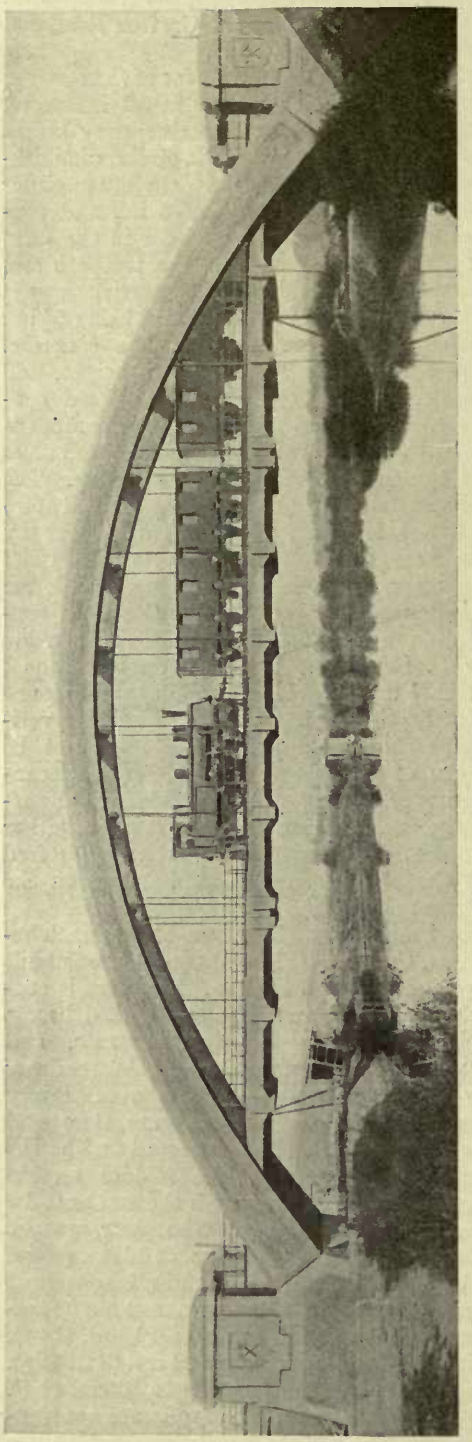


Fig. 503.—Bridge over River Werra, Thuringia

gravel ballast is 30 cm. to 40 cm. (12 in. to 16 in.) deep. A photographic view on p. 203 shows the very interesting type of centering employed, this being of parabolic form, with a maximum excess of height in the centre of 50 mm., diminishing towards the end. The supports rested on wedges, of which the central ones were the first to be driven out. During the concreting the centering settled 35 mm. at the crown of the arch, and when the centering was removed after the concrete had hardened for five weeks there was a further settlement of 11 mm. The loading tests, which were applied on January 20, 1911, the work having been begun on April 15, 1910, were conducted with three loaded railway trucks, each weighing about 48 tons, and a locomotive weighing 34 tons, and the results indicated a modulus of elasticity of 300,000 kg. per square centimetre (roughly, 1,904 tons per square inch).

Walnut Lane Bridge, Philadelphia (*see* Fig. 506), was designed by George S. Webster, M.Am.Soc.C.E., in conjunction with Henry H. Quimby, M.Am.Soc.C.E., respectively chief engineer and assistant engineer to the Department of Public Works, Philadelphia. It crosses the Wissahickon Creek and ravine, and connects Germantown and Roxborough, the steepest grade of approach to the bridge being $4\frac{1}{2}$ ft. in 100 descending from Germantown. The roadway is 40 ft. wide between the kerbs, the side walks being each 8 ft. wide. The main arch is 233 ft. clear span between the piers, the deck of the bridge being 147 ft. above the creek and the underside of the arch 133 ft. above the creek. The springing point of the arch at the face of the piers is 34 ft. above the drive. Economy and lightness are secured by designing both the main arch and the bridge arches in the form of twin supporting arch ribs floored across the top. Over each main arch rib the floor is carried on eight piers of spandrel or relieving arches, which support the four longitudinal walls upon which the floor will rest. The whole is hollow underneath clear up to the floor, and as no earth filling is required, there is a great saving in weight. Each of the five bridge spans is 53 ft. clear between the piers, and those next to the main span are about 70 ft. clear above the ground.

A bridge near Teufen, Switzerland, is illustrated by Fig. 507. The following particulars of it are reproduced from an article in *Concrete* by Willy Obrist,

who was in part indebted to an article in the *Schweiz. Bauzeitung,* by Prof. Mörsch, the designer of the bridge. The contractors were Froté Westermann and Co., Zürich. The bridge, which replaces an iron bridge erected in 1856, crosses the Sittertobel in one large arch, 79 m. span and 26·5 m. rise, which is succeeded on the left bank by four arches 10·25 m. in span and on the right bank by two such arches (Fig. 507). The roadway is about 66 m. above the level of the river, and has a width of 6·9 m. between the parapets, including two footpaths, each 60 cm. wide. The main span is treated as an arch with fixed ends and without hinges. As live load, a steam roller of 20 tons weight and a crowd amounting to 450 kg. per square metre (92 lb. per square foot) were considered, provision also being made for a variation of temperature of 20° C. each way. The form of the arch is that of the curve of equilibrium for the dead load. The thickness at the crown being 1·20 m. and at the abutments 2·13 m., the maximum and minimum stresses were calculated to be 31·3 and 0·4 kg. per square centimetre (445 and 5·7 lb. per square inch). Although no tensile stresses should theoretically occur, yet in order to secure a proper distribution of the loads due to the spandrel columns, the arch was reinforced with rods of 28 mm. diameter,

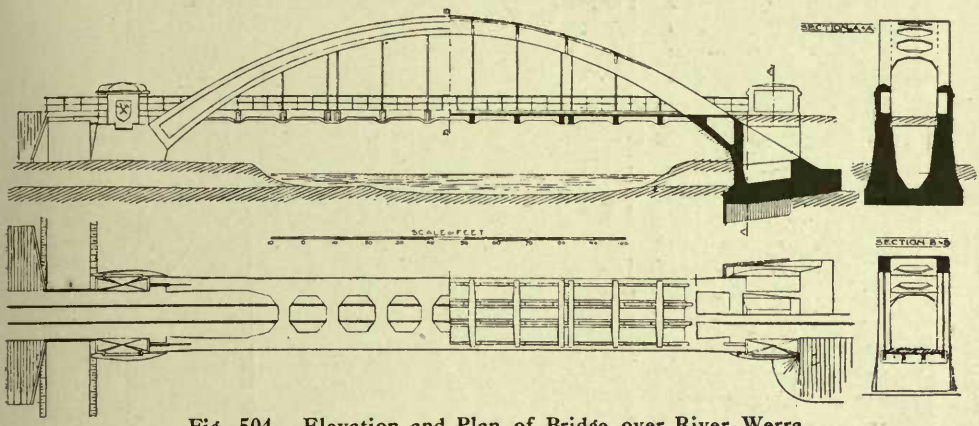


Fig. 504.—Elevation and Plan of Bridge over River Werra

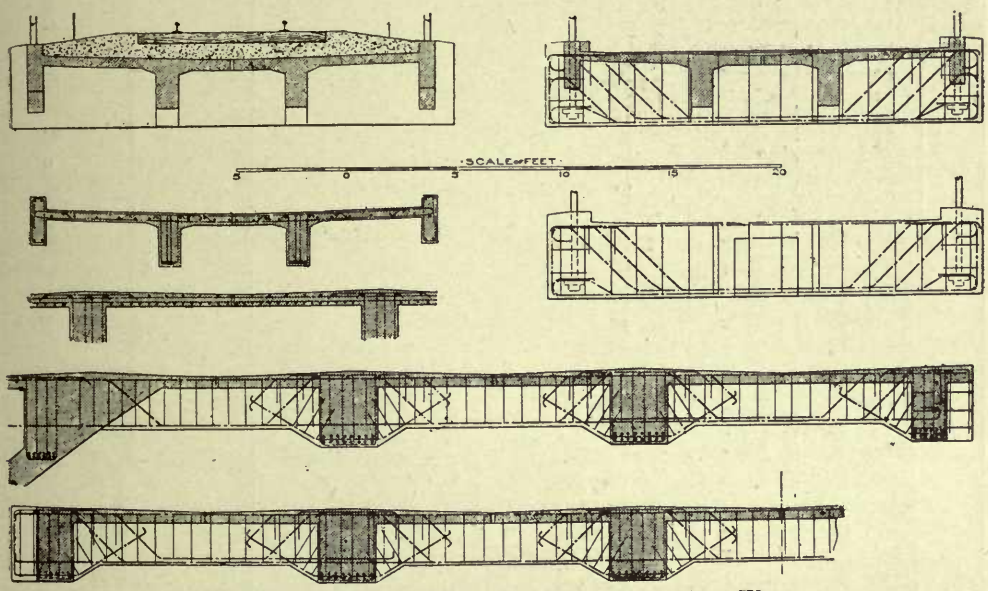


Fig. 505.—Details of Beams for Bridge over River Werra

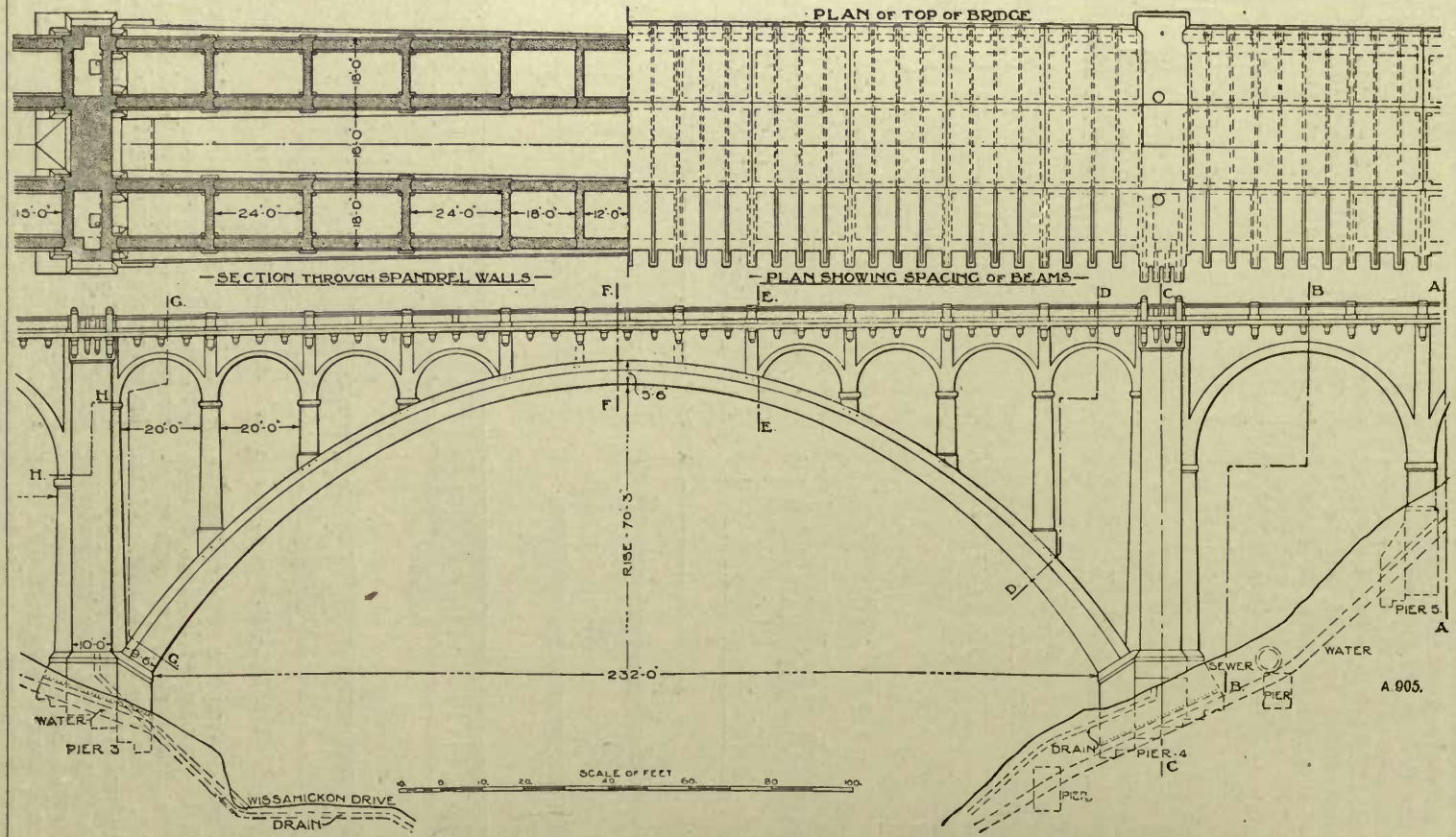


Fig. 506.—Plan and Elevation of Walnut Lane Bridge, Philadelphia

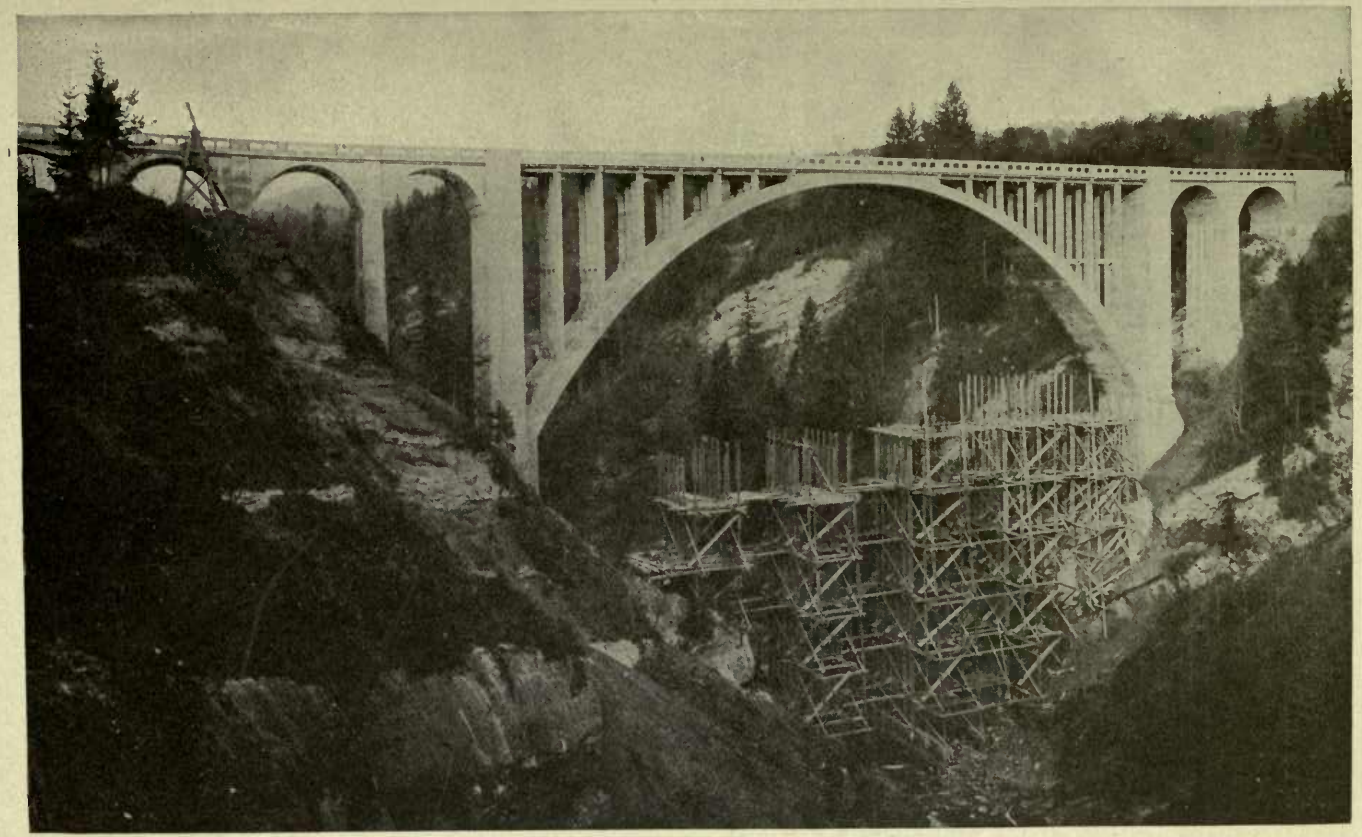


Fig. 507.—Bridge near Teufen, Switzerland

there being at the crown ten such rods above and eighteen below, in the quadrants fourteen above and below, and at the abutments eighteen above and ten below. The breadth of the arch is 6·5 m. at the crown, and this gradually increases, in order to secure lateral stability, to 7·50 m. at the abutments. The foundations are in interstratified marl and sandstone, and sustain a maximum load of 6·5 kg. per square centimetre (92 lb. per square inch). The roadway consists of a reinforced plate 20 cm. thick and four longitudinal beams, and is supported from the arch by columns of 50 cm. by 50 cm. section. The beams are treated as continuous, and are respectively 1·49 m. and 1·37 m. apart, 4·5 m. span, 65 cm. high, and 25 cm. broad. The connection with the main columns is made by means of a so-called "pendulum wall," 25 cm. thick, the elasticity of which allows it to follow all the deformations of the roadway due to changes of temperature. The smaller arches are also so formed that only compressive stresses occur; all are reinforced with five rods of 14 mm. diameter, the thickness at the crown in each case being 35 cm.

The centering used was a model of its kind (*see* p. 200). The timbers were so arranged as to be subject only to compression, all loads being directly borne. Channel irons were placed between the vertical and horizontal timbers to distribute the loads. The six ties, 1·4 m. (4 ft. 7 in.) apart, were bedded on sand boxes at a height somewhat above the springing of the arch. As the timbering stood without load from December, 1907, to the following March, it was necessary to provide for proper resistance to wind pressure. Special struts were therefore placed perpendicularly to the ties, supported below at 10 m. to the right and left of the axis of the bridge. The good construction of the centering is clear on consideration of the deflections observed. During the concreting a depression of 32 mm. ($1\frac{1}{4}$ in.) occurred, and on removal of the centering a permanent deflection of 5 mm. ($\frac{3}{16}$ in.) was found.

The construction of the work was begun in March, 1907, and the bridge was taken over on November 7 of the next year.

The concrete for the reinforced portions was composed of a mixture of 1 yd. gravel, $\frac{1}{2}$ yd. sand, and 600 lb. portland cement.

Varied Applications and Examples of Reinforced Concrete

THE ROYAL AUTOMOBILE CLUB'S BUILDING

Excavations and Retaining Walls.—The Club House of the Royal Automobile Club, erected in Pall Mall from the designs of Messrs. Mewes and Davis and E. Keynes Purchase (the joint architects), in 1909–11, undoubtedly ranks as one of the finest buildings in London, and the many and varied examples of reinforced concrete work comprised by it give it an interest, from our point of view, surpassed by no other building in London. The site, 228 ft. by 140 ft., was

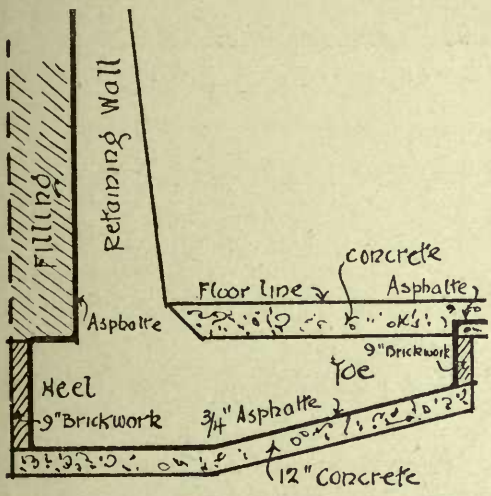


Fig. 508.—Retaining Wall with Asphalt Damp-proof Course

formerly occupied by the old War Office buildings, adjoining the Carlton Club.

Excavations, etc.—The soil throughout is a good London ballast, which, although virgin soil, required careful strutting at all parts in order to prevent it running down; and one feature of the work was the very heavy timbering, some idea of which can be gathered from the photographic views. Water was reached at a level of about 30 ft. below the pavement line, and as some of the foundations had to be carried to a depth of more than 47 ft., two pumps were constantly at work in order to keep the water below the level at which the operations of excavating and concreting were being executed. This was done by sinking a pit at the lowest level to which the water drained, and then continuously pumping to prevent the water rising above a certain height. The second pump kept the trenches dry and prevented any portion becoming flooded.

In a few cases the foundations are carried right through the overlying ballast down to the London clay, this great depth being necessary as there are two floors below the pavement level; and owing to the high level of the sewer in Pall Mall all the low-level drains are carried to two pneumatic ejectors which raise the sewage to the required height. These ejectors are placed in a specially constructed chamber at a depth of 20 ft. below the basement floor, and in order to prevent any risk of sliding, the grillage foundations under any adjoining stanchions are carried down to the same depth as the ejector chamber. Owing to the presence of water, and the basement floor being below the level of the sewer, the whole building is erected in what constitutes a huge underground tank, an asphalt damp-proof course (*see* Fig. 508) being continuous and carried under and around all grillages, walls, chambers, and subways of every description, so as to prevent water from any quarter penetrating the building after completion. The horizontal asphalt under floors was $\frac{1}{2}$ in. thick, and under foundations or where vertical $\frac{3}{4}$ in. thick; the horizontal asphalt under all floors is covered with 12 in. of cement concrete in order to prevent it being raised by the water pressure; the head of water was not great enough to justify the reinforcing of this concrete.

Retaining Walls, etc.—An interesting feature of the work is the construction of the retaining walls, which are excellent examples of plain concrete and reinforced concrete work. The manholes in some cases are so deep that the walls had to be calculated as retaining walls; in several instances the

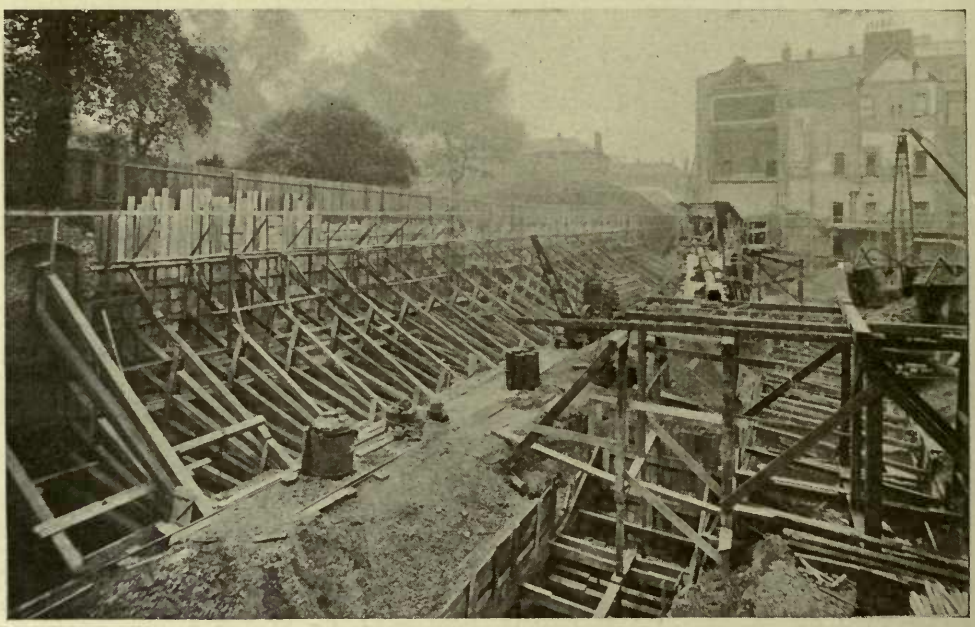


Fig. 509.—Timbering to Support Earth, before Building Retaining Wall (R. A. C.'s Building)

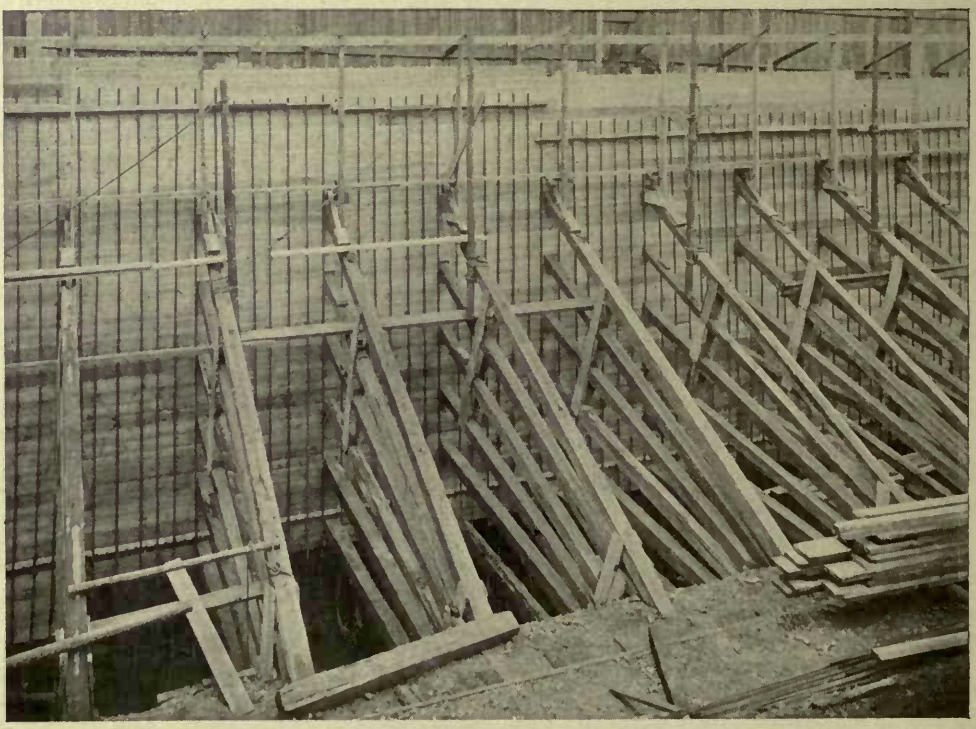


Fig. 510.—Shuttering and Reinforcements for Retaining Wall (R. A. C.'s Building)

walls are 3 ft. 6 in. thick at the base. The retaining wall at the rear of the site is of reinforced concrete, 200 ft. long and nearly 40 ft. high, and supports Carlton Gardens, which adjoin the site on the south. Fig. 509 shows the rear of the site as well as the timbering erected to support the earth before the wall was put in. The other trenches shown are for the grillage foundations of the stanchions carrying the main back walls of the building. The great width of the trenches can be seen and also the timbering that was required, this consisting of poling boards, waling pieces, and struts where above water level; whilst below that level, runners were driven down as the work proceeded in order to keep the earth back.

The reinforced wall, although of such great length and height, is only 3 ft. to 3 ft. 6 in. thick at the base, with a vertical face against the gardens and a battering face next the new building. The whole is constructed of portland cement concrete, machine-mixed, in the proportion of 1, $1\frac{2}{3}$, $3\frac{1}{3}$, reinforced with indented steel bars varying from $\frac{1}{2}$ in. to $\frac{7}{8}$ in. in thickness. The vertical bars are carried with various curves into the heel of the wall, which projects under Carlton Gardens, and into the toe on the opposite side of the wall; and are tied together with horizontal bars throughout. Some of these bars are of such a length and shape that it was found impossible to cart them to the site in the form required, and therefore they were bent on the works by a hydraulic press, three bars being shaped at a time.

Fig. 510 shows the shuttering for the wall and some of the vertical bars in position. Near the top of this view will be noted some horizontal pieces of wood, which are notched in order to receive the various bars and keep them in position, and also to maintain sufficient space between the bars and the shuttering so as to ensure the former being covered with concrete in the finished wall. Small pieces of brick are placed under the bottom of the bars with the same object. Although not necessary for purposes of strength, the horizontal and vertical bars are wired together where crossing each other, the object being to ensure that the reinforcement shall not be moved out of position during the filling in of the concrete.

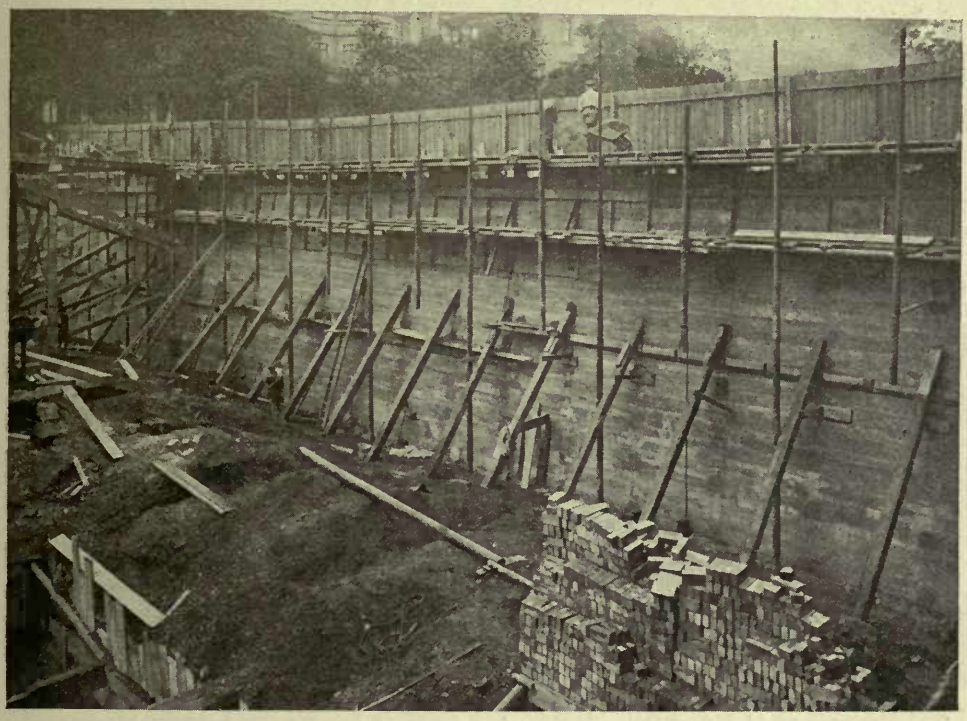


Fig. 511.—Retaining Wall Complete (R. A. C.'s Building)

This is certainly the best method of carrying out the work, although in this instance it meant some 15,000 wirings. These walls and foundations were executed by Messrs. Trollope, Colls and Sons, Limited, of Grosvenor Road, Pimlico.

A view of the completed reinforced retaining wall is shown by Fig. 511; the strutting was removed in sections, the bottom row being taken away first, and then the second row, and so on until the whole had been dispensed with. The first tier was removed six weeks after the concrete had been rammed in position, and each hole was carefully filled up with concrete immediately the strut had been taken away. An interval of seven days was allowed to pass between the removal of each row in order that the weight coming upon the wall might be gradually applied.

A good comparison may be made between the back and front retaining walls. The front wall forms the outer wall of the vaults under the pavement and supports the roadway in Pall Mall, and is executed in plain cement concrete (1, 2, 4). The total length is 228 ft., and the height 25 ft. Upon calculation, the thickness of the base was found to work out at 6 ft., and as the space in the front vaults was not quite so important as that in the rear of the main building it was decided to use plain concrete. Had reinforced concrete been employed, a much thinner wall could have been constructed, as the back wall is 40 ft. high and only 3 ft. 6 in. thick at the base, whilst the front wall is only 25 ft. high and yet needs a thickness of 6 ft. at the base. At the same time, of course, it must be borne in mind that the front wall has much heavier work to do owing to the roadway which it supports. The wall has a vertical face on the inside, and on the outside, next the roadway, has two offsets, the first of which reduces the thickness to 4 ft. 6 in., and the second to 2 ft. 6 in. There is no heel to the wall, but a toe is formed at the base which projects under the vaults to the extent of 2 ft. 6 in. More than 30,550 cubic feet of concrete were required in the construction of this wall, but the work was speedily carried out, owing to its simple character, although great care had to be exercised in providing temporary supports for the roadway and the numerous gas, water, telephone, and electric light mains met with.

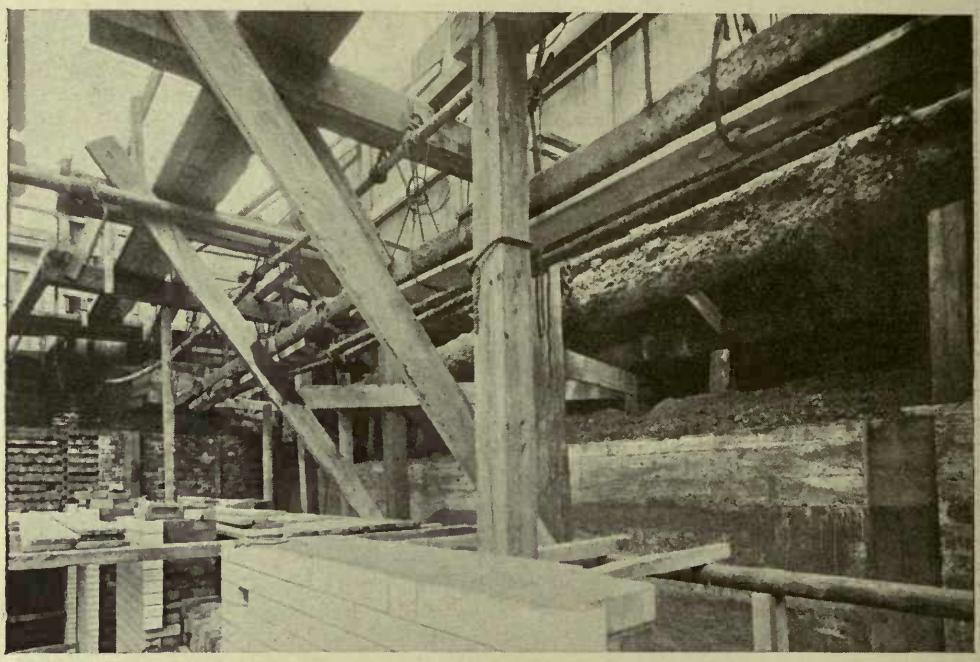


Fig. 512.—Inside Face of Retaining Wall, and Methods of Supporting Mains (R. A. C.'s Building)

Fig. 512 shows the inside face of the front retaining wall, after the shuttering had been removed, and the various methods that were employed for holding up the mains that come over the vaults. It also shows the wall of the vaults that faces the club house (there being an area along the front of the building

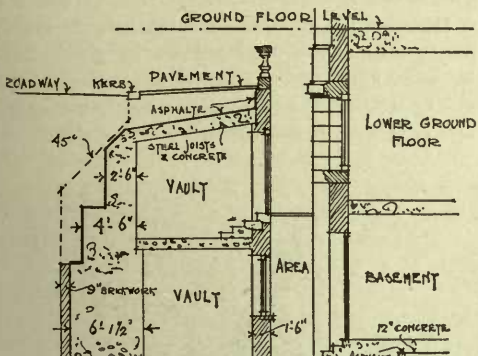


Fig. 513.—Section showing Front Vaults (R. A. C.'s Building)

3 ft. 6 in. wide), which is built in white glazed bricks for the sake of cleanliness and light. The asphalt damp-proof course, previously mentioned, is carried over the walls and roofs of the vaults, passing under the retaining wall, and up on the outside of

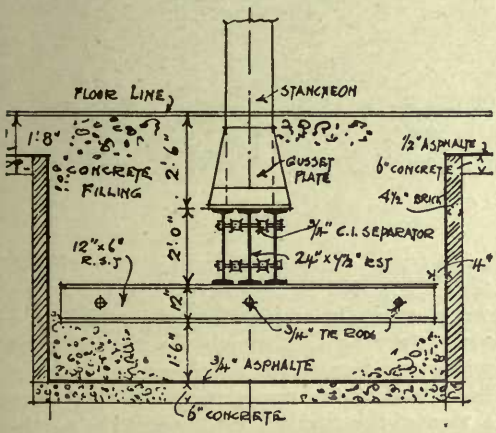


Fig. 515.—Section through Top Layer Beams of Grillage

it, being finished under the pavement, against the stone balustrade, which forms a finish to the vault wall, as shown in Fig. 513. The sloping roof over the vault is constructed of rolled steel joists and concrete and is provided with openings for coal shoots, etc.

Foundations.—Before describing the foundations and other work below the base-

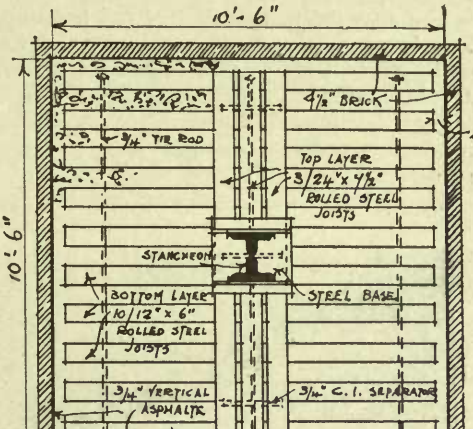


Fig. 514.—Plan of Grillage Foundation (R. A. C.'s Building)

ment floor level, it will be interesting to give some idea of the accommodation provided on this floor.

The central space is occupied by an elliptical vestibule, 48 ft. by 32 ft., from which the various corridors lead to the other

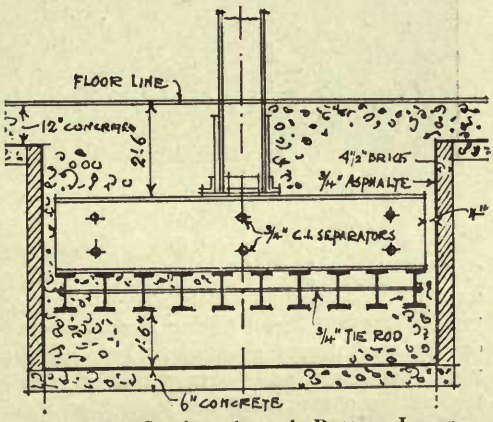


Fig. 516.—Section through Bottom Layer Beams of Grillage

portions of this floor, and the main staircase leads to the ground floor. On the east side is a Turkish bath, and on the west are three racquets courts and a gymnasium, the latter being connected with a swimming bath,

which occupies the rear of the building. The front of the building on this floor provides private dressing-rooms for the members and a large dining-hall for the servants, while the boiler-house for heating purposes is also situated in this part of the building. The space in the front vaults is used for coals, stores, large workshop, and lavatories. In addition to this accommodation there are several lavatories, dressing boxes, locker rooms, attendants' rooms, and a large servery.

The grillage foundations (Figs. 514 to 516) are carried to a great depth in order to obtain a good bottom below any adjoining drains, underground flues, and pits of various kinds. The building is constructed on the steel frame principle, where all the walls, partitions, and floors (with the exception of the party walls) are carried on steel girders at each floor level, the total weights being brought down by the stanchions alone, which, consequently, carry very heavy loads.

There are as many as 112 stanchions which actually run down to the soil, whilst there are a few more which are carried by large plate girders higher up the building. The loads on the stanchions, of course, vary considerably, but in some cases over 500 tons has to be carried, and consequently it will be seen that an ordinary plain concrete foundation would have been so large and the thickness so great as to be almost impracticable. It was decided to use grillage foundations, and, although this is quite a common form of construction in large buildings, a detailed explanation is given here because they are a typical example.

The grillages are so large that, although the stanchions are spaced at fair distances apart, one continuous trench from 12 ft. to 15 ft. wide is necessary to execute the work, as can be seen in Fig. 517, which is a view showing the trench for the grillages under the stanchions which carry the main back wall of the upper part. It also gives a view of the site looking towards the north-east corner, and in the distance can be seen a full-size model of one bay of the proposed new front elevation, which was erected in wood, in order that a true conception of the finished building might be obtained with regard to detail and proportion.

Fig. 517.—Trench for Grillages under Stanchions (R. A. C.'s Building)

The plan of the grillage foundation shown in Fig. 514, on the preceding page, is repro-

duced from an actual drawing of one foundation, and may be regarded as a typical case. The safe load put upon the soil can be taken approximately at about 3 tons per square foot, and, consequently, as the foundation proper measures 10 ft. 6 in. by 10 ft. 6 in., it may be assumed that the load carried by this particular stanchion is 110 sq. ft. multiplied by 3 tons=330 tons. This weight has to be brought down and spread over the required area of soil by a stiff foundation. Figs. 515 and 516 show how this is accomplished by placing the base of the stanchion directly on to three 24-in. by $7\frac{1}{2}$-in. rolled steel joists, which are tied together and the webs stiffened by three pairs of $\frac{3}{4}$-in. cast-iron separators and $\frac{3}{4}$-in. bolts. The webs are further stiffened against any buckling by the application of angle stiffeners (not illustrated) $3\frac{1}{2}$ in. by $3\frac{1}{2}$ in. by $\frac{5}{8}$ in. under the base of the stanchion, twelve of these being used in each case. The steel joists forming the top layer of the grillage act as cantilevers on two sides of the stanchion base and distribute the load to the second layer, which in this particular instance is composed of ten 12-in. by 6-in. rolled steel joists spaced slightly over 12 in. centre to centre. These joists are tied together with three rows of $\frac{3}{4}$-in. tie rods, with pieces of gas barrel between the webs to keep the joists rigid; these latter act as small cantilevers on each side of the top layer and distribute the load over the concrete and soil beneath. Under the joists is placed

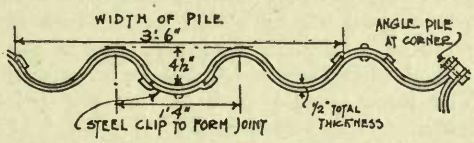


Fig. 519.—Section of Steel Pile

cement concrete 18 in. thick, and the whole of the joists and stanchion base are filled in with and surrounded by concrete.

Before placing the grillage in position, a pit was formed with a concrete bottom 6 in. thick and brick sides $4\frac{1}{2}$ in. thick, lined with an asphalt damp-proof course as illustrated in the drawings. Below the water level, the bottom and sides of this pit are increased to 12 in. and 9 in. respectively. The view given in Fig. 518 shows the asphalt damp-course as it was applied under the base of the back retaining wall. The excavations between the grillages, where these are close together, are filled in with gravel well rammed (not

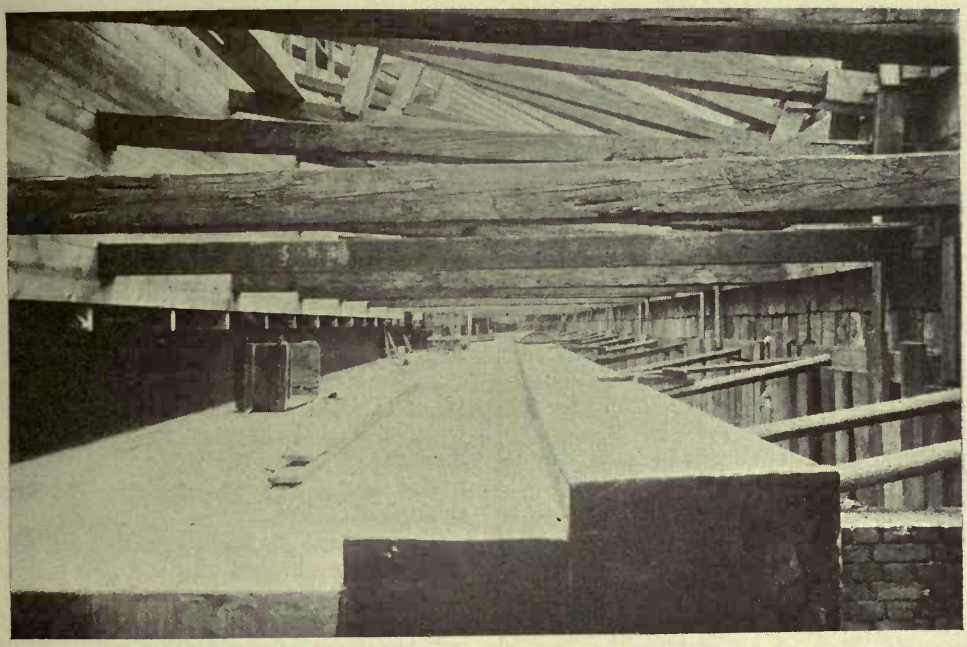


Fig. 518.—Asphalt Damp-proof Course under Base of Back Retaining Wall (R. A. C.'s Building)

with concrete) in order to keep each grillage foundation separate and prevent any unequal loading or settlement.

Steel Piling for Foundations. — Although various forms of steel piling have been used extensively in the construction of sea walls, docks, collieries, and such work, where great strength and permanence are required, steel piling for foundations is quite a modern feature in London building work. The form of pile used in this particular instance is indicated in Fig. 519, which is a horizontal section showing the piling and the method of forming the angles. The piles are of corrugated steel $\frac{1}{4}$ in. thick, but the lap given to each pile is so great that in the finished work the thickness of the piling in every case is equal to $\frac{1}{2}$ in. Thus, if it were necessary to obtain finished steel piling, 1 in. thick, $\frac{1}{2}$-in. piles only would be

Fig. 520.—Top of Finished Steel Piling

used, and the lap employed as shown. The piles are 3 ft. 6 in. wide, with corrugations about $4\frac{1}{2}$ in. deep and 1 ft. 4 in. centres, the object of the corrugations being, of course, to increase the lateral strength of the piling. The length of the piles employed is 15 ft., and Fig. 520 is a view taken about 15 ft. below the basement floor level, showing the top of the finished piling along one side of the chamber.

The driving of the steel piles is effected

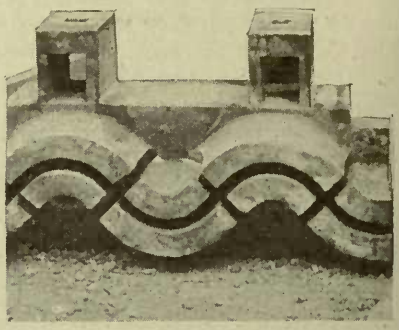


Fig. 521.—Cast-iron Cap to Head of Steel Pile

in a similar manner to that employed for ordinary timber piling, but a very heavy monkey is used (as the piles are not pointed in any way), weighing $1\frac{1}{2}$ tons, and this is given a drop which varies from 1 ft. to 4 ft. The head of the pile is protected by a specially made cast-iron cap (*see* Fig. 521), weighing about 3 cwt.; the under side of the cap is shown in this view, and its application to the pile is obvious. The greatest distance that a pile is driven by one blow is $\frac{1}{2}$ in., and great care is exercised during the driving to prevent the pile being turned out of its course by any obstruction it may meet. The vibration caused by the monkey is considerable, owing to its great weight. When the four sides of the pilings are complete, the whole of the excavation can be carried out within the piling, and the new work executed without any further timbering or supports.

The makers of this form of piling have now brought out one of a different section, considered to be superior to the earlier one.

Floors.—The floors and roofs throughout the Royal Automobile Club's building are of reinforced concrete, with the exception of the main girders, which are of steel alone. The principle is that of a steel framed building, with the floor panels filled in with reinforced concrete, the latter being a separate construction, which receives its support from the main steel girders; but it cannot be said that the whole floor forms one homogeneous structure, as occurs when the main girders and panels are constructed of reinforced

concrete at the same time. Generally speaking, the steel girders are fixed at a uniform level of 4½ in. below the floor line to the top flange, while the top of the concrete is finished 3½ in. below the floor line; thus a covering of 1 in. of concrete is obtained over the main girders; and this allows some of the reinforcing bars to be carried over these, as can be seen in Fig. 522, which shows the reinforcement and centering in position previous to the filling in of the concrete.

The small wire clips on the main steel girder support the concrete casing, which protects the lower part, and are quite enveloped in the concrete filling when the latter is complete. It will be noticed that every third bar is carried over the steel girder and allowed to run for some distance into the adjoining panel, thus to a certain extent making the floor continuous and preventing cracks from occurring over the girder, as would otherwise be the case, and at the same time giving some provision against shear near the girder. Where the concrete floor occurs on one side of the girder only, the reinforcement is cranked well over the top flange and thus hooked to the girder. Fig. 522 shows that a certain amount of reinforcement is provided at right angles to the main reinforcing bars. These are bars ⅓ in. square spaced at 2-ft. centres, to form a tie and distribute temperature stresses which might cause cracks.

The type of reinforcement used was the patent indented steel bars of ½-in. and ⅓-in. section, and the tests imposed were sufficiently severe to ensure the steel being free from any brittle tendency. The size and spacing of the bars, of course, varies according to the span, which, generally speaking, ranges from 8 ft. to 14 ft., but as a general rule the distance from centre to centre is 6 in., while the thickness of the concrete varies from 4½ in. to 7 in. The ground and lower ground floors were calculated to carry a load of 220 lb. per sq. ft., and the upper floors and flat roofs a load of 160 lb. per sq. ft. The concrete was composed of 1 part portland cement, 2 parts sand, 2 parts clinker, and

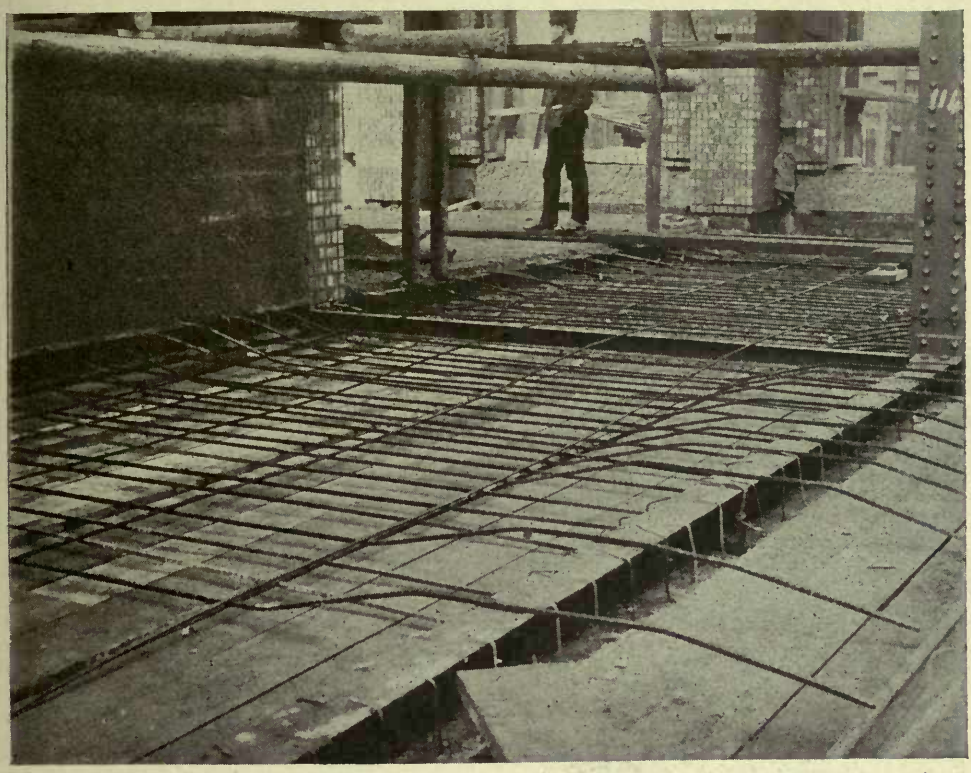


Fig. 522.—Reinforcement and Centering for Floor (R. A. C.'s Building)

2 parts ballast or broken brick, and the whole was mixed by machinery.

The cement was required to comply with the British standard specification for medium-setting cement, with the exception that the tensile strength was to be at least 600 lb. per sq. in. after twenty-eight days' setting, instead of 500 lb., as called for in the British standard. The sand was screened to pass a $\frac{3}{16}$-in. mesh, and contained a mixture of fine and coarse sand; this was mixed with the cement in the dry state before the addition of the coarse aggregate. The gravel was screened and crushed to pass a $\frac{3}{4}$-in. diameter ring, and the clinker was required to be broken to the same state and be free from all coke breeze, ashes, or coal. The whole of the operations of crushing, screening, gauging, and mixing were accomplished in a specially erected machine at one point, immediately under efficient supervision, and from this point the concrete was rapidly transported to the different parts of the works as required; and in no case was more than half an hour allowed to elapse between the time of mixing and the time of placing in position. Where new work was placed next to the surface of existing concrete the latter was cleaned of all loose particles and dust, and given a thick coat of neat cement grout before the new concrete was rammed in position. Care was taken to strengthen all portions of the floors, where small holes were required for pipes and similar purposes; and where the holes were large proper steel framing was provided.

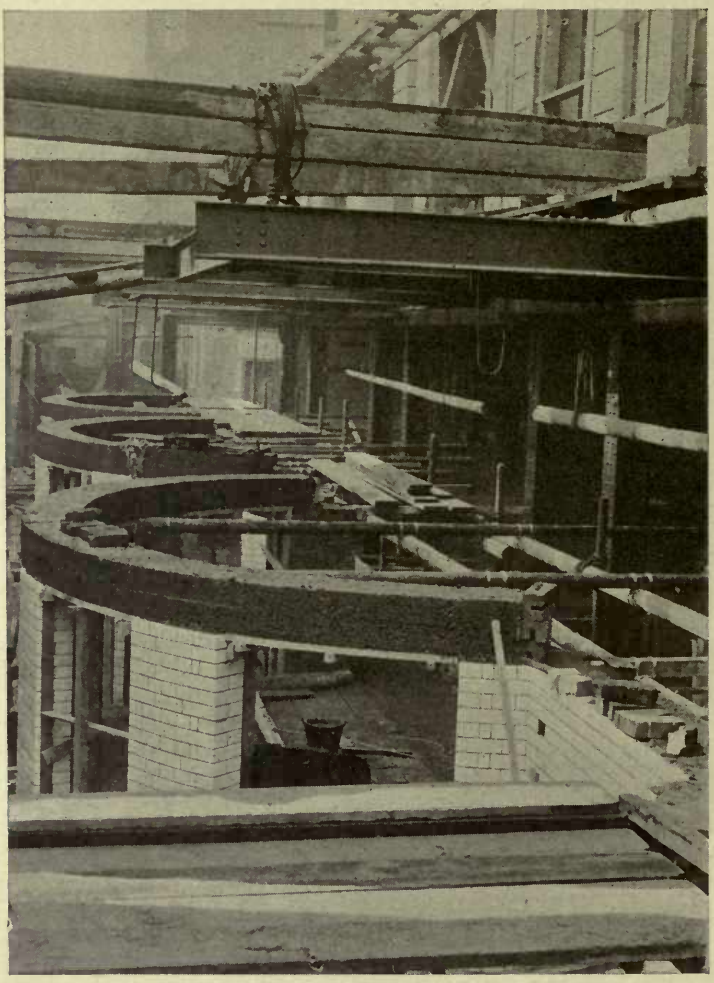


Fig. 523.—Elliptical Reinforced Concrete Lintels

Lintels, Ceilings, etc.—Fig. 523 is a view showing the reinforced concrete lintels which occur over the window openings in the three large elliptical bays to the swimming bath in the basement floor. The steel framing to the terrace above at the ground-floor level was erected as shown and suspended in position until the walls were built up to receive same. In addition to the floors and roofs, which are all in concrete, special suspended concrete ceilings are constructed in the majority of the rooms on all floors. These are required owing to the varying size of the main girders, which project below the concrete floor and the numerous ventilation ducts, pipes, and cables carried about in all directions, thus rendering it quite impossible to form a satisfactory

ceiling line without special construction. These ceilings are formed of pumice concrete slabs, 2½ in. thick, and they are suspended from the concrete floor and girders above, with steel hangers at intervals which carry inverted steel tees at their lower ends, on which the slabs rest, wire lathing being provided to give a key for the plaster on the under-side of these steel tees.

Staircases.—The staircases provide some very interesting forms of construction, both in the nature of their planning and the materials employed in their execution, which vary greatly with the importance of the staircase. In a building of this size many staircases are necessary, and there are no less than fourteen in all, exclusive of the small flights which lead to the terraces, and others similar. The form of construction employed is, generally speaking, that of concrete steps, strengthened by the addition of steel joists, channels and tees, but reinforced concrete, wood, and plain concrete are also used in different cases.

The main staircase is that leading from the ground to the fifth floors, and this is situated close to, and leading from, the large elliptical vestibule, 48 ft. by 32 ft., which occupies the central portion of the ground-floor plan. The staircase well on the upper floors is 27 ft. 6 in. long and 16 ft. 6 in. wide between the walls, while on the ground floor it is 43 ft. 6 in. long and terminated by a semicircular end. The flights themselves are 5 ft. 6 in. wide in the clear, and the treads are 12½ in. wide with a rise of 6 in., while the lowest steps, which land on the ground floor, are gradually increased in width until the bottom tread is 18 in. wide, and these lower steps are also shaped in curtail fashion to discharge everyone toward the centre of the staircase hall and away from the wall. The stairs are formed in concrete with rolled steel joists at each half landing and floor level, to support the 10-in. by 3½-in. raking steel channels, the latter carrying the concrete stringer and the outer ends of the steps; the inner ends being supported by 14-in. brick walls surrounding the staircase itself. These raking

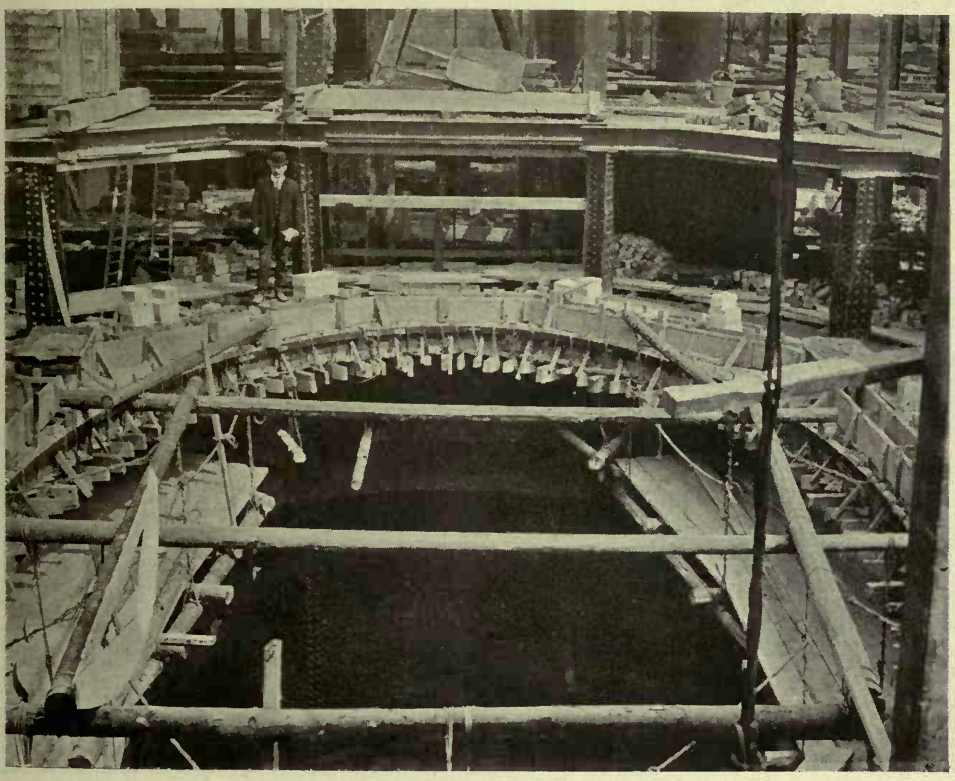


Fig. 524.—Shuttering to Curb of Skylight over Elliptical Vestibule (R. A. C.'s Building)

channels are cleated and riveted to the steel joists with all necessary ramps, and the steps themselves are further strengthened by the addition of 2-in. by ½-in. steel tees, which form a core, and ¾-in. steel tie rods which serve to tie the channels to the walls. No projections such as beams are allowed to show below the soffit, as all the steelwork is entirely enveloped in concrete sufficiently thick to give one continuous flowing line, which follows the pitch of the stairs, while the concrete stringer, which is about 2 ft. 2 in. deep and projecting 2 in. above the steps and the same distance below the soffit, is also continuous and carried round all landings and floors with a graceful curve. This necessitates the use of more concrete than would be necessary if the beams were allowed to project below the soffit, but the effect obtained is a very pleasing one, and the idea of graceful and easy ascent is conveyed to everyone looking up the staircase. The handrail is carried by an iron balustrade fixed to the concrete stringer and following the same curves. The landings, treads, and risers are finished with Eschaillon stone, 2 in. and 1½ in. thick, this being a very hard cream-coloured French marble which is used unpolished, while the walls and the stringers are finished in stucco, closely resembling stonework.

Of the other staircases, that running from the basement to the fifth floors will not be used for much continual work, service lifts being provided for the upper floors, and consequently it is constructed of concrete, on the cantilever principle, each step obtaining its support from one wall only, with a certain amount of bearing on the step below, an open well between the up and down flights being maintained. The concrete used is composed of granite chippings and portland cement in the proportion of 4 to 1, and the steps, which are 2 ft. 9 in. wide, are strengthened by the addition of steel tee cores. They are finished with Mason's patent treads and 1¼ in. of granolithic.

Fig. 525.—Shuttering and Reinforcements for Sloping Roof (R. A. C.'s Building)

Reinforced concrete is employed in two

instances on account of its adaptability under peculiar conditions. One of these cases is that of a staircase which leads from a lobby adjoining the lounge on the ground floor to a restaurant placed on the first floor, the shape being very winding and curious, and as such great difficulty would have occurred in shaping ordinary steel channels and joists.

The other instance is that of a small staircase 2 ft. 6 in. wide which has a support on one side only, and in addition has to pass over a large ventilating duct where every inch of space was of great value. A portion of the steps required to be designed as cantilevers, and these are reinforced with two ½-in. indented steel bars in their upper surfaces, while a portion of the flight itself was required to act as a beam where passing over the duct previously mentioned. This necessitated some of the treads forming an abutment for the beam portion, and these were reinforced with three ½-in. bars instead of two, as the longitudinal reinforcement was so designed to distribute the pressure over three treads in each case. The bottom flight, which is about 12 ft. long, was designed to act as a beam without any appreciable support at the sides, ½-in. bars spaced at 3-in. pitch being provided longitudinally for the full length of the flight with transverse bars at 12-in. centres, while the minimum thickness of the concrete was 5 in. The remaining staircases are constructed of steel framing enveloped with concrete, and no detailed description is necessary.

Central Elliptical Vestibule.—Fig. 524 illustrates the work over the large central elliptical vestibule, 48 ft. by 32 ft., which occurs on the ground floor. This is carried right up through the first floor, with a gallery around at this level, and finished midway between the second and third floors with a large skylight and a ceiling light under. Owing to a large cove which was required on the inside, all the steel framing had to be bent to the required curve, and this rendered the work very complicated. The photograph is taken above the second floor, and shows the shuttering for the concrete curb to the large skylight.

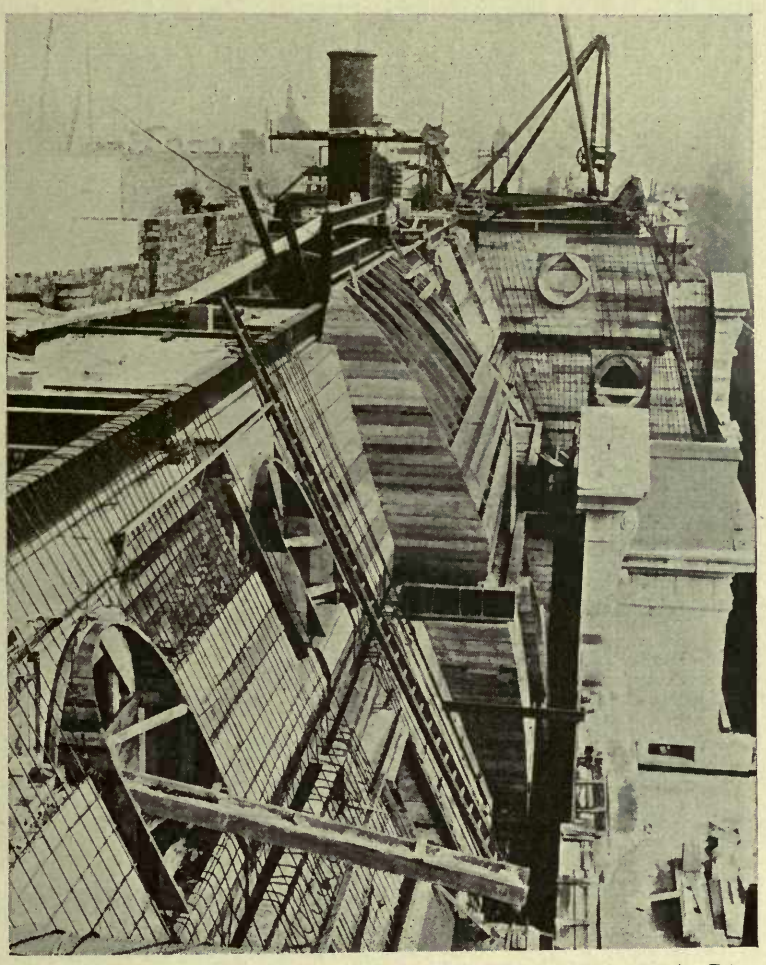


Fig. 526.—Shuttering and Reinforcements for Sloping Roof (R. A. C.'s Building)

Sloping Roofs.—Sloping roofs occur on both of the elevations. Fig. 525 shows the shuttering and reinforcement in position, and a portion of the concrete during the execution of the roof facing Pall Mall. The fourth floor only comes within the roof at this point, and the height is 10 ft. 6 in. from the floor level to the asphalt flat above. The pitch of the roof is practically 60 degrees, and it is carried through in one straight line for the entire length of 228 ft. without any break as shown. The thickness of concrete employed is $4\frac{1}{2}$ in., and the reinforcement consists of $\frac{1}{3}$-in. indented steel bars placed vertically at 10-in. centres, these being turned over the rolled steel joists, which act as trimmers at the intersection of the flat and sloping portions of the roof. They are also secured in a similar manner to the steel joists placed at the foot of the slope. In addition to the vertical reinforcement, $\frac{1}{3}$-in. bars are placed horizontally at 2-ft. centres to distribute temperature stresses.

Numerous openings are provided at regular intervals throughout the length for the dormer windows, these being formed in such a manner that the windows are set back within the roof and not allowed to project in the usual manner. This necessitated forming the cheeks and the small flat at the bottom of each window in concrete also, and the same thickness is employed as in the main slope. The cross bars are all turned round into the cheeks and, in addition, vertical bars are provided. The small flat at the bottom of each opening is reinforced with $\frac{1}{3}$-in. bars at 10-in. centres, and similar bars, two in number, are placed at the head and bottom of the opening in the sloping concrete to act as a tie, and round these are bent the vertical bars which are cut off on account of the opening. An excellent idea of the shuttering adopted can be obtained from Fig. 525.

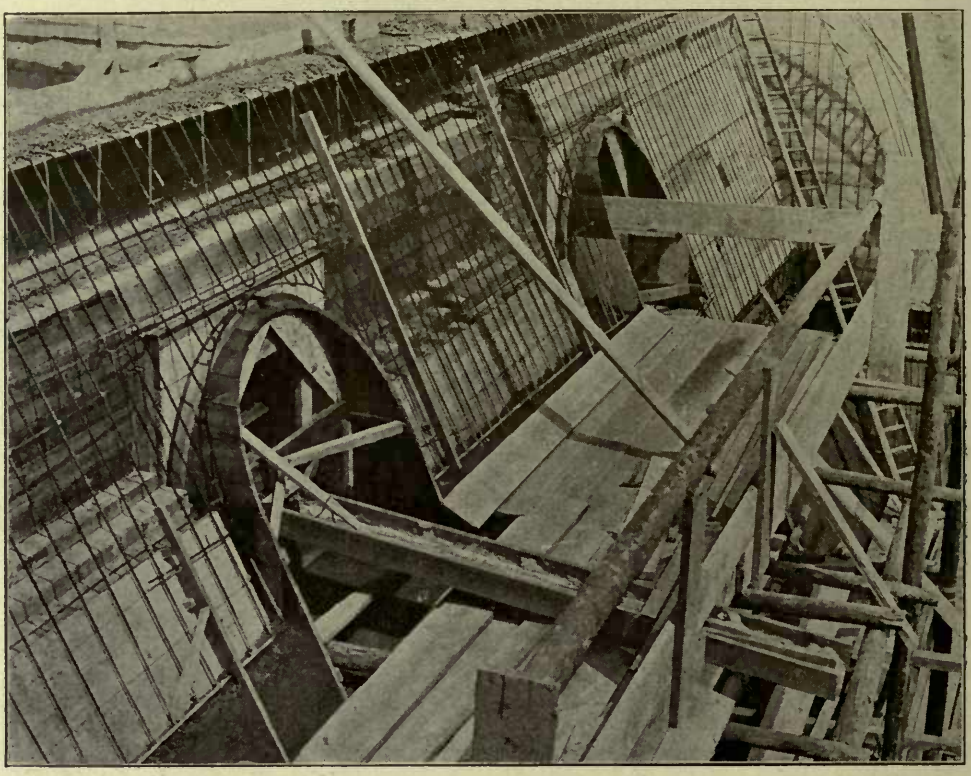


Fig. 527.—Reinforcement around Elliptical Openings in Sloping Roof (R. A. C.'s Building)

The work to the back portion is of a slightly more complicated nature, owing to the fact that two floors occur in the roof, thus the length of the slope is considerably more. The two floors are the fourth and the fifth, and the total height is 20 ft. 6 in.

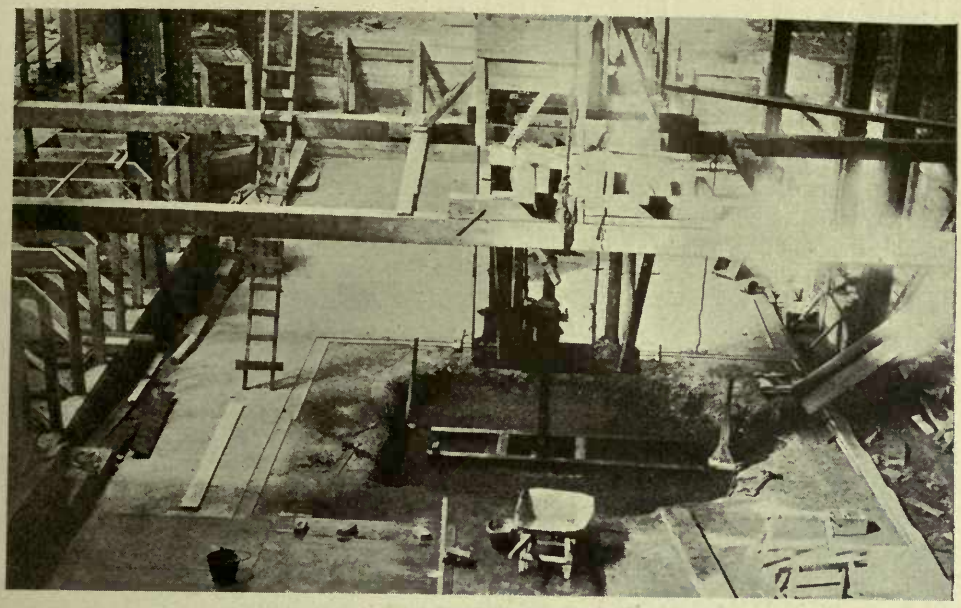


Fig. 528.—Shuttering for Swimming Bath

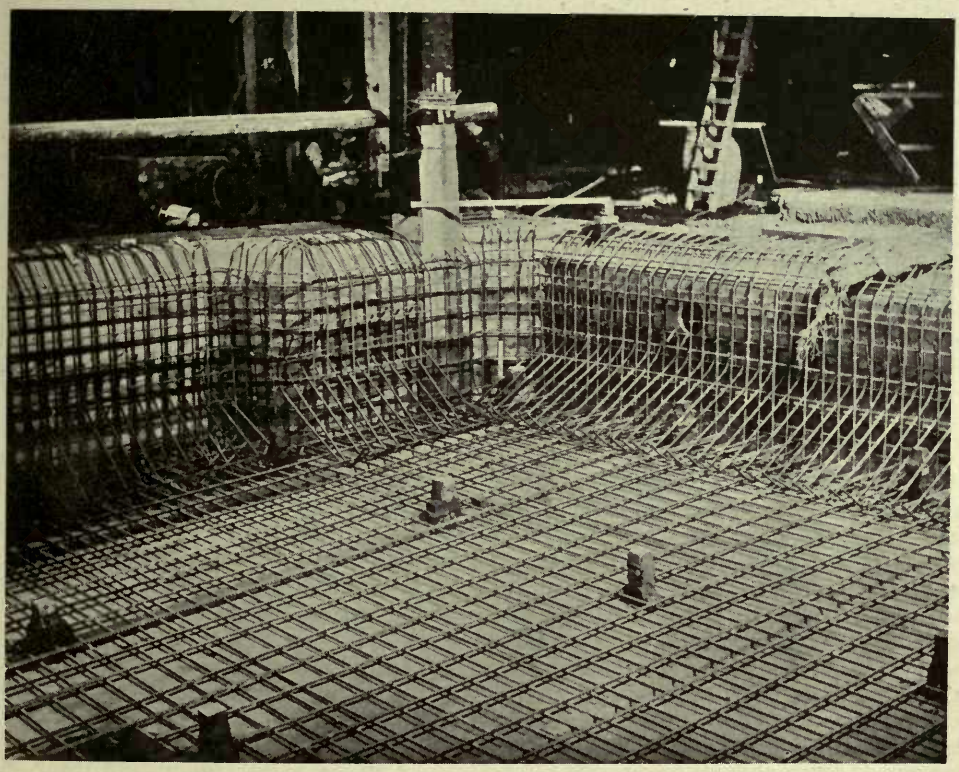


Fig. 529.—Reinforcements for Swimming Bath (R. A. C.'s Building)

from the fourth floor level to the finished roof level. It will be seen, however, on referring to Fig. 526, that steel framing is introduced at the fifth-floor level, and thus the long slope is cut up into two portions, although every alternate bar is made continuous. This roof, unlike that facing Pall Mall, is not continuous, but is divided up by one central and two end pavilions, which rise above the general roof level about 4 ft. 6 in. The central pavilion, which is curved in section, and one end pavilion are noticeable in Fig. 526. The concrete to the central pavilion is built on the curve, and a portion of the shuttering for this can be seen. The general part of the roof between the pavilions is built to a pitch of 65 degrees and the end pavilions to a pitch of 75 degrees. Another feature which entailed careful execution is the introduction of elliptical and circular openings for the windows, and a detail view of the work around two of these openings is shown in Fig. 527. The reinforcement is hooked over a curved bar which follows the line of the opening. The various bars are secured at the crossings to prevent displacement when concreting.

The concrete was composed of 1 part portland cement, 2 parts sand, 2 parts clinker and 2 parts ballast, mixed by machinery, and the whole operation of crushing, screening, gauging and mixing was accomplished at one point in a specially erected machine.

Swimming Bath.—The construction of the swimming bath affords an excellent example of the adaptability of reinforced concrete to work of all kinds. The bath is situated in the central portion of the south side of the basement floor, and its size is 86 ft. 9 in. by 30 ft. 8 in., these measurements being to the face of the concrete work only. The depth varies from 4 ft. 4 in. at the shallow end to 8 ft. 10 in. at the deepest point, the latter being at a distance of 18 ft. from the diving end, which is 7 ft. 4 in. deep, thus giving a maximum depth of water at the point required by the bather when diving from the highest stage. These measurements are from the floor level to the

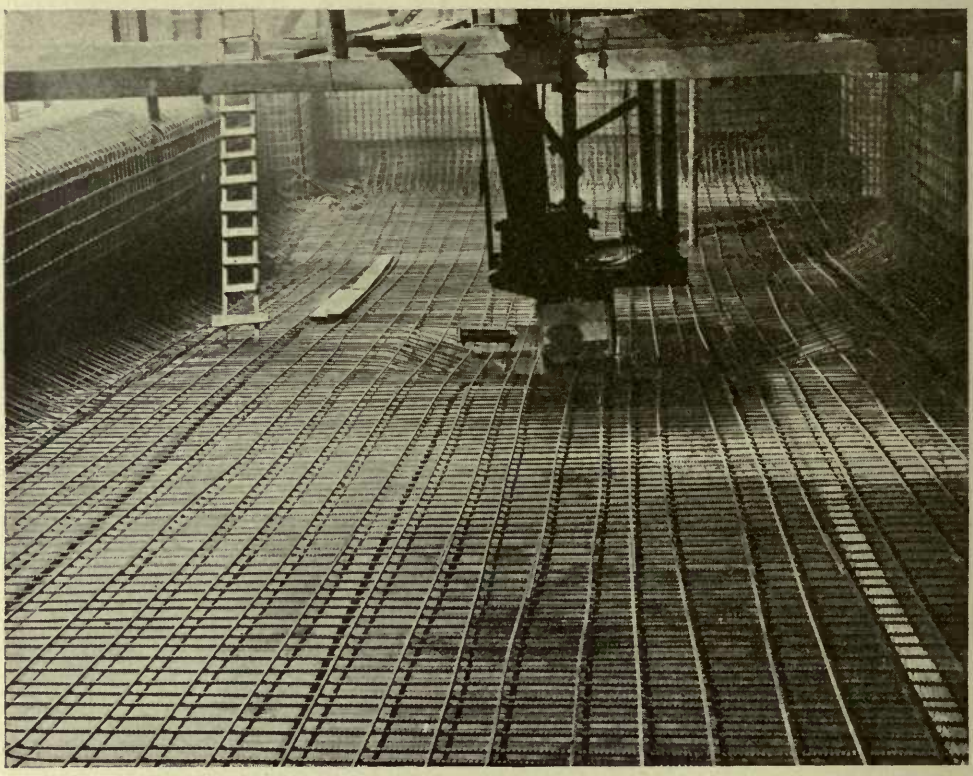


Fig. 530.—Reinforcements for Swimming Bath (R. A. C.'s Building)

bottom of the bath, the water level being 10 in. below the floor line.

Two flights of steps are provided on each side near the ends, and these do not project into the bath in the ordinary way, but are recessed in the sides in order to keep the entire surface of the bath free from obstruction. The reinforced wall is carried round on the outside of these steps, and the latter are carried on plain concrete stepped as required.

One of the principal points considered in the constructional work to the bath was that of keeping the side and end walls and the bottom entirely free from the adjoining stanchions and grillages, in order to avoid the possibility of any settlement of the latter from affecting the bath, especially as this would be liable to crack the marble lining and cause serious defects. There are twelve stanchions which run down at different points round the bath, and these are situated close to the edge, as will be seen from the fact that the distance from centre to centre across the bath is only 33 ft. 9¼ in., while the width of the bath itself in the rough is 30 ft. 8 in., and thus thick walls could not have been adopted such as would have been necessary with brick or plain concrete.

Another point in favour of reinforced concrete was the fact that a subway was required around all four sides of the bath on the outside, and this was limited in width by the many grillages and other underground work in the vicinity making it necessary to retain every inch of space that was possible, and this subway prevented the wall of the bath from receiving any assistance on the outside, and, in addition, necessitated carrying the reinforced floor over the subway.

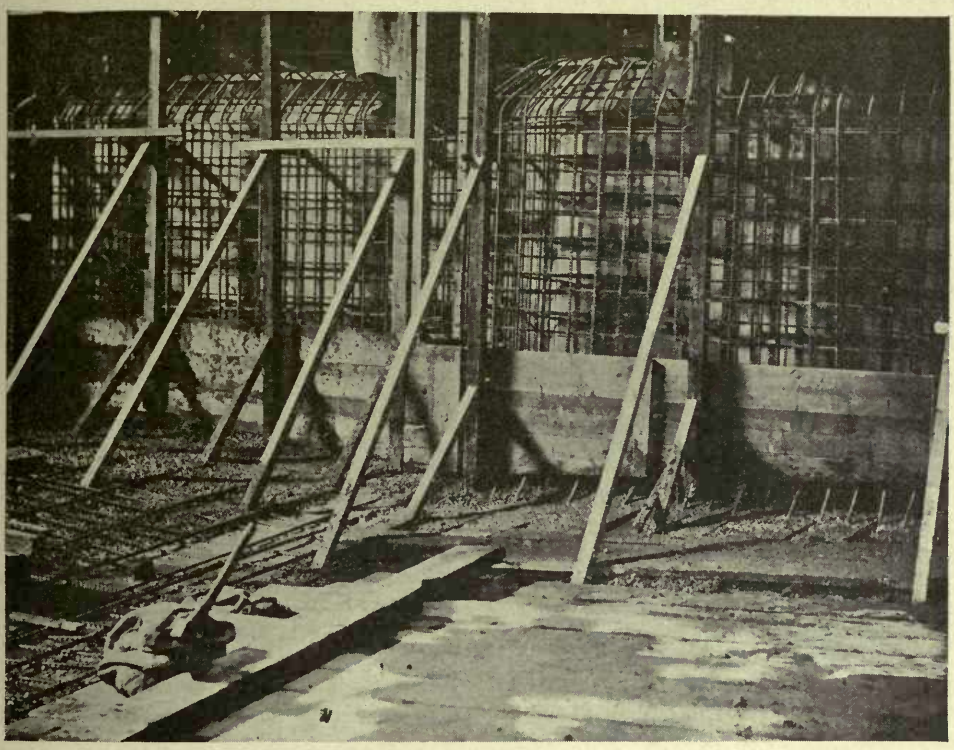


Fig. 531.—Reinforcing Sides of Bath and Concreting First Layer of Bars in the Bottom

The whole of the space to be occupied by the bath and subway was first prepared, and the continuous asphalt damp-course, which is carried under the entire building, was formed and laid to the required levels to suit the work of the bath proper.

Fig. 528 shows a portion of the shuttering for the concrete work being erected, and a portion of the subway is just noticeable. The last stage of the pumping is also depicted, with the asphalt damp-course completed with

the exception of the small piece surrounding the well, from which the water is being drawn. This well was formed to allow the pump to work and keep the surface water from flooding the work, as the bottom of the bath is below the water level, and the pumping was continued until the last possible moment. When the suction pipe was removed, the pipe leading to the well was sealed with a special cap, the concrete filled in, and the asphalt damp-course made good, thus completing the huge underground tank in which the entire building stands.

After the asphalt was complete, the concrete work was begun, and carried out as rapidly as possible. The thickness of the side and end walls is 12 in., while 18 in. of concrete is employed for the bottom. The concrete is splayed at the junction between the vertical and horizontal work, and it is also splayed back at the top of the walls.

To prevent the necessity of forming a hole in the reinforced work and in the asphalt damp-course below for the outlet to the bath, which is situated at the deepest point, a dip or trench is formed in the two former to allow the water pipe to be carried through to the valve chamber above them, and the work is brought to the level as required with plain concrete filling. The reinforcement to the bottom consists of $\frac{5}{8}$-in. square indented steel bars spaced at 6-in. centres across the width of the bath, and similar bars longitudinally at 12-in. centres. These are placed in the under surface of the concrete, and a similar amount of reinforcement is employed in the upper surface.

The reinforcement of the walls consists of vertical bars $\frac{1}{2}$ in. square, spaced at 6-in. centres, with the same size bars placed horizontally at 9-in. centres. The splayed portions are also reinforced, and the manner in which this is done is clearly illustrated in Fig. 529, which is a view showing the first layer of reinforcement in position at the shallow end of the bath. An excellent idea of the work in progress can be gathered from this photograph, indicating, as it does, the shuttering, the reinforcement in position, the recess for the steps, and the small blocks used for levelling purposes. The hole which is shown through the shuttering at the end of the bath is one of the four openings required for the inlet pipes. The bars were securely wired at every crossing, this entailing about 20,000 wirings in all, the object being to prevent any possible displacement of the reinforcement during the filling in of the concrete. The concrete was composed of 1 part portland cement, 2 parts sand, and 4 parts of ballast; and the lower layer of bars was first concreted in the bottom, and then 1 ft. of the sides, returning again to the bottom when the second layer was encased. After this, the sides were completed, and during the whole time occupied in the execution, the whole of the concrete in position was watered night and day in order to form one solid mass when complete, and avoid the possibility of the work being in separate layers without proper cohesion.

Fig. 530 gives a good idea of the reinforcement to the whole bath, looking from the shallow end, and it shows the bars as placed approximately in position previous to the wiring together at every crossing. Fig. 531 illustrates the reinforcement to the sides and shows the concreting of the first layer of bars in the bottom, and the shuttering for the lower part of the sides.

The total weight of the reinforcement used in the bath is about 40 tons. After the completion of the concrete work, a layer of asphalt was applied in the inner surface, to form a water-tight lining and prevent any possibility of the water in the bath from percolating through the surrounding work. The finished bath is lined throughout with white Sicilian marble, $1\frac{1}{2}$ in. thick, laid in slabs, 3 ft. wide and the full depth of the bath, without horizontal joints. The scum trough and spittoons are in similar marble, as are also the floor paving around; outlets from the first-named are taken through into the subway previously mentioned, and carried into a drain running round the bath. The steps leading to the bath have marble treads and risers.

Procedure in Executing the Work.—The following is a brief description of the procedure in executing the reinforced concrete at the Royal Automobile Club. The cement upon arrival at the site was tested. Careful attention was paid to the selection of store, and a clean, dry vault was used, with facilities provided for covering up in frosty and mild weather. The bags were only released in the quantity required from time to time, and not left lying about the site. The aggregate for the reinforced floors and roof was as follows: 1 cement, 2 sand, 2 broken brick, 2 clinker.

The broken brick was of approved quality, not soft, all traces of lime and dirt removed

and screened to pass a $\frac{3}{4}$-in. mesh, the dust resulting from the crushing being removed. The clinker was carefully inspected for impurities and live matter (such as sulphur and coal), and if not well burnt was condemned. No coke breeze or ashes were allowed on the job. The sand was clean and sharp, and passed a $\frac{3}{16}$-in. mesh. For the small amount of concrete which was mixed by hand, gauge-boxes were measured and cubed, and, if found accurate, were marked, and only those so marked were allowed to be used. The clinker, broken brick and sand having been gauged, the cement was thrown over the mass, the whole was turned twice dry, water added by means of a rose watering-can and then turned twice wet, the water being taken from the company's main. Most of the concrete used on the job was mixed by machine.

Fig. 532 is a diagram showing the concrete-handling and conveying plant in use on the Royal Automobile Club building. The ballast and bricks were shot from the skips of the steam crane to the staging of the crusher. The aggregate from the crusher was shovelled into the revolving 2-in. mesh screen, falling thence into a hopper below, the outlet of which was directly over the gauge-box, the last-named being worked on a centre pivot. Upon lifting two iron shutters fitted at the base of these outlets, the ballast and clinker fell into the gauge-boxes, the pin was withdrawn, and the material precipitated into the mixer below. The sand and cement were gauged on the staging and shovelled into the mixer. The mixed concrete was shot into a skip on the lift hoist, and raised to the required floor, by which method the whole of the building was concreted from the one spot.

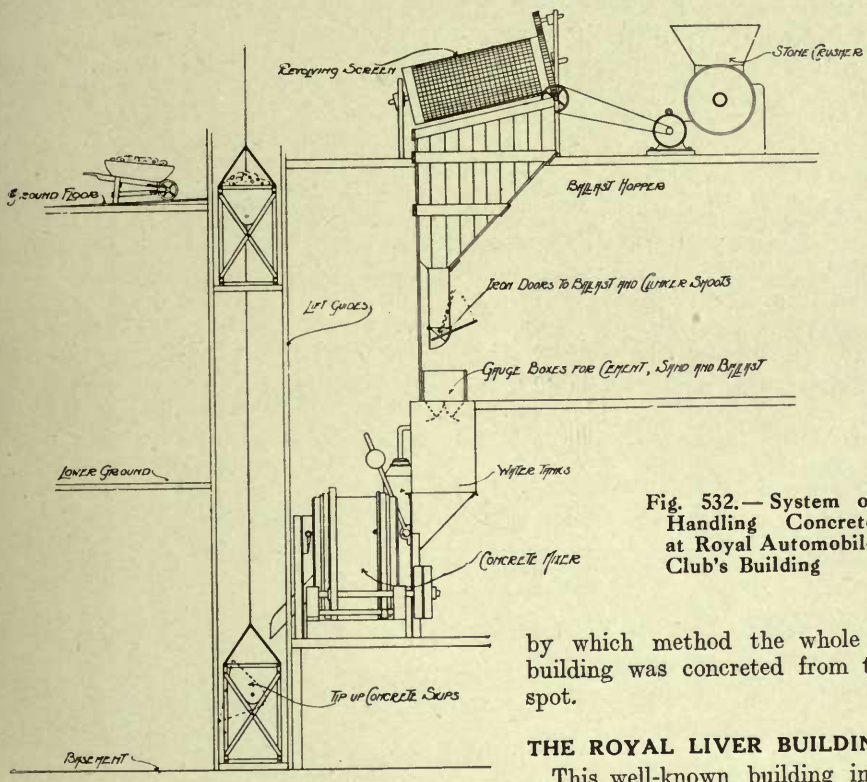


Fig. 532.—System of Handling Concrete at Royal Automobile Club's Building

THE ROYAL LIVER BUILDING

This well-known building in Liverpool is constructed entirely of reinforced concrete, and may be regarded as the first "skyscraper" erected in England. The site, which was formerly occupied by St. George's Dock, is about 301 ft. long and 175 ft. 6 in. wide. The architect was Mr. W. Aubrey Thomas, of Liverpool, and the Hennebique system of construction was employed. The height from the street level to the main roof is about 167 ft.; to the centre of the clock, 226 ft.; and to the top of the Liver bird is 310 ft. There is a basement floor 12 ft. below the street level, while the depth of the foundations varies from 18 ft. to 40 ft. below the floor level, giving a total average

distance of 351 ft. from the foundations to the top of the tower. There are eleven stories in the main building, and seventeen stories in each main tower. There has been much discussion at various times as to the relative advantages of reinforced concrete and structural steel when rapid construction is an important consideration, and as an example of the possibilities of the former, it is interesting to note that in this building, which covers an area of about 45,000 sq. ft., the first seven floors were constructed in an average time of nineteen working days per floor. Very large spans were adopted in order to give the maximum amount of clear floor space, and this necessitated the carrying of very heavy loads by the columns. The foundations were formed by large concrete piers, which were sunk to the level of the old red sandstone, and connected at the top by reinforced concrete slabs. The site is surrounded by retaining walls, which in reality act as deep beams spanning the distance between the foundation piers, the latter having an average distance of 30 ft. apart. The subsoil water level actually rises to a height of 10 ft. above the basement floor line, and the walls were formed with 5-in. panels only, stiffened by vertical counterforts at short intervals apart. A continuous coping, 18 in. wide and 8 in. deep, was constructed at the top of the wall, and the base consisted of a wide toe and a slightly projecting heel. This portion of the work was very difficult on account of the water. In some cases, the foundation piers were not available as supports for the retaining wall, and cantilevers were then formed which projected 11 ft. from the main columns and carried the wall, together with the basement flooring.

Fig. 533.—Columns and Arched Ribs in the Royal Liver Building

The columns provide some interesting

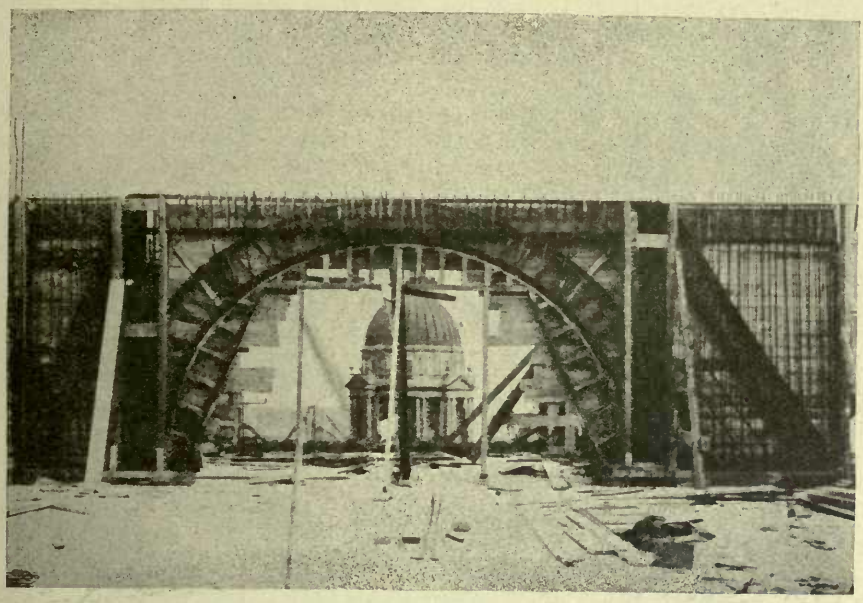


Fig. 534.—Arched Beam (before Concreting) in Royal Liver Building

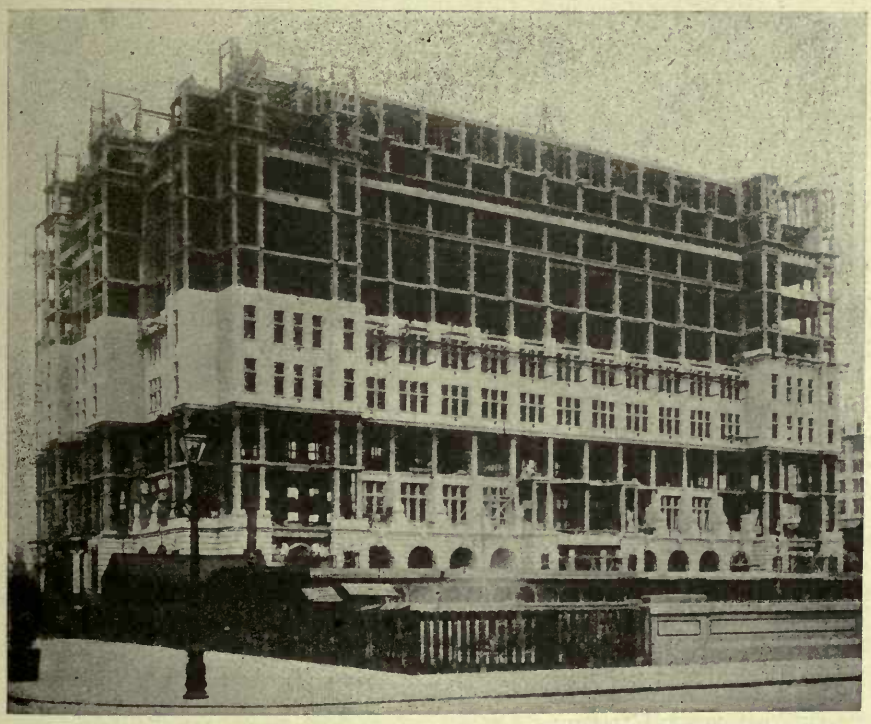


Fig. 535.—"Skeleton" of Royal Liver Building

construction on account of the exceptionally heavy loads to be carried; the interior ones beneath the main towers sustain a total load of 1,200 tons each, while the corresponding columns against the external walls support a total load of 1,500 tons each. These columns carry a load of 900 tons at the ninth floor level, and they are 3 feet square at this point, while in the basement they have an extreme size of 8 ft. by 3 ft., with reinforcement consisting of twenty-five bars $1\frac{3}{4}$ in. in diameter, and sixteen bars $1\frac{7}{8}$ in. in diameter, tied in every direction by $\frac{1}{4}$-in. diameter steel links spaced at 3-in. centres. The remaining columns in the exterior walls, which carry the granite curtain walls and the floor loads, support a total load of about 750 tons each at the lowest level, where they are 8 ft. by 1 ft. 9 in., while the average size at the top is 14 in. by 12 in.

The floor beams, generally speaking, have a span of about 40 ft. with a width of 9 in. and a depth of 34 in., including the thickness of the floor slab. The reinforcement consists of four bars in the tensional area and two bars in the compressional area, provision against shear and diagonal tension being made by turning up two of the tension bars at the ends of the span and by the introduction of vertical stirrups. Some large beams were required at the ninth and tenth floor levels in the west main tower, these having spans over 40 ft. long in some cases, and in addition support is given to two large arched beams which each carry 1,480 tons. These arches have a clear span of 60 ft. and a rise of 32 ft., thus extending from the ninth floor level to 2 ft. 9 in. above the eleventh floor level. The thickness of the concrete is only 1 ft. 9 in., and they form an interesting example of scientific construction which would be impossible in any other material. Other examples of arched members are provided in the glass roof over the ground floor. The clear span is 58 ft., and the total depth required was only 23 in., and the width 7 in., this giving the maximum amount of glass area, and allowing the construction of members which are slender and in keeping with their function. The floors were constructed throughout of the Hennebique hollow tubes, which are admirably suited for a building of this description, where efficiency, combined with lightness, are essential. These tubes are formed of concrete or fireclay, and they are made about 20 in. wide by 24 in. long by 7 in. high, and are placed in parallel longitudinal rows between the secondary beams, the latter being moulded in position between the tubes, which are temporarily supported on planks carried at each end by the main beams. The tubes are formed with sides which are inclined slightly outwards towards the top, thus preventing any tendency to drop out on the under-side of the floor, and the top of the tubes are

Fig. 536.—Cathedral at Manila, Philippine Islands

slightly arched to increase their strength. A minimum covering of 1½ in. of concrete was employed above the tubes, thus forming a floor construction 8½ in. thick. This system has the advantage that no elaborate centering is required, as the sides of the tubes form a permanent shuttering for the secondary beams, while the tubes can, if necessary, be made on the site, thus avoiding any possibility of delay.

The external walls throughout are constructed with a granite facing which, generally speaking, has a thickness of 14 in., and this is carried at each floor level by the reinforced concrete construction. This method enabled several floors to be built simultaneously, thus saving a great deal of time in the finishing of the structure.

The towers are finished with domes constructed in reinforced concrete and covered with sheet copper, while large circular openings were required for the clock dials which are slightly larger than those in the Clock Tower of the Houses of Parliament. Many interesting staircases are formed in the building, and all of these are in reinforced concrete, those in the upper part of the towers being of the spiral form. Figs. 533 to 535 are photographic views which give some idea of the magnitude of the building.

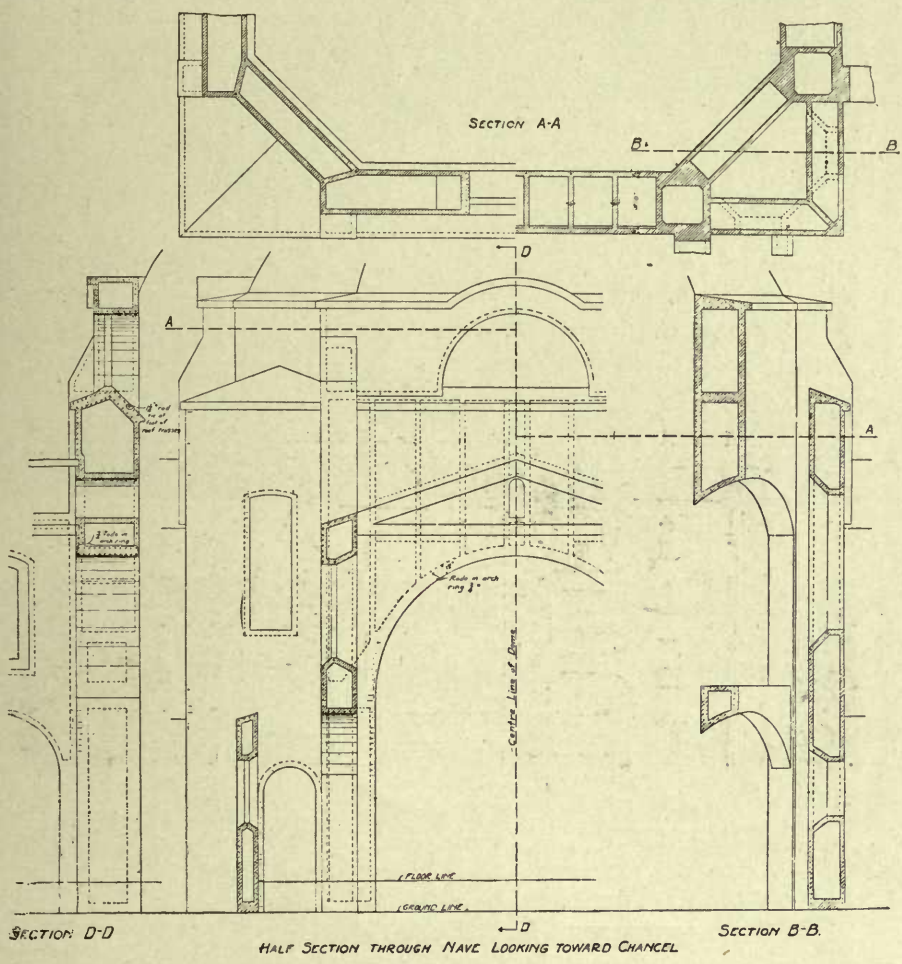


Fig. 537.—Various Sections through Nave of Manila Cathedral

CATHEDRALS AT MANILA AND POTI

Figs. 536 to 545 show the Episcopa Cathedral of St. Mary and St. John erected at Manila, in the Philippine Islands, the

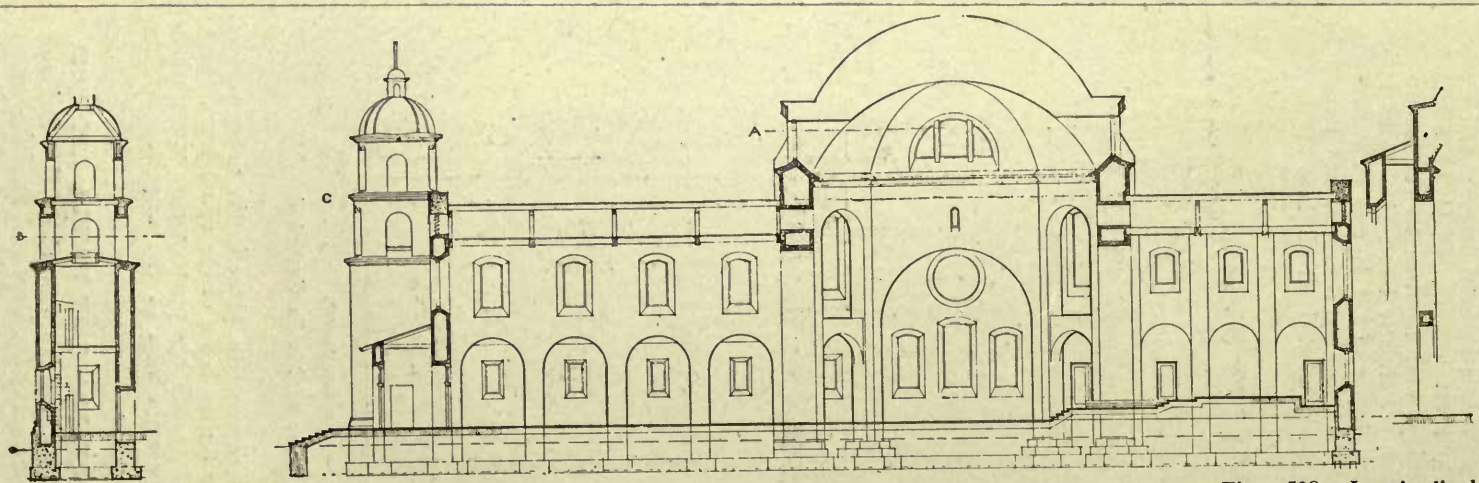


Fig. 538.—Longitudinal Section (looking North) and Details of Manila Cathedral

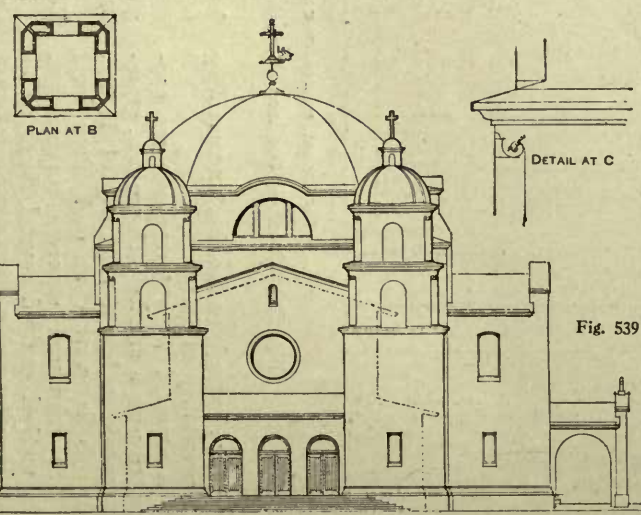


Fig. 539

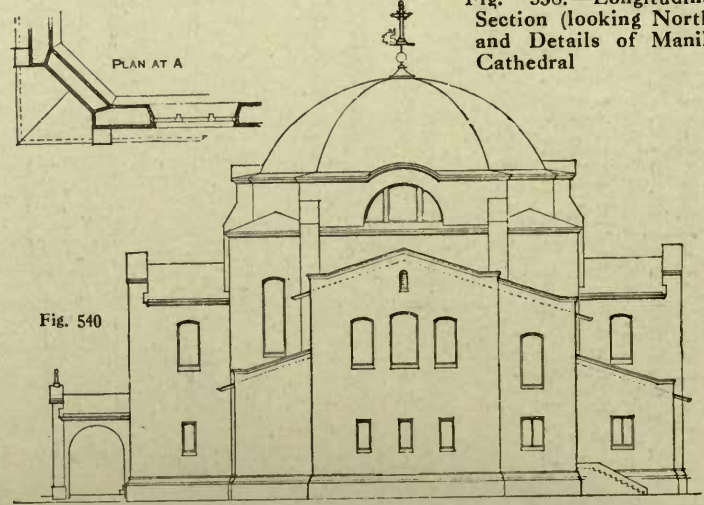


Fig. 540

Figs. 539 and 540.—West and East Elevations of Manila Cathedral

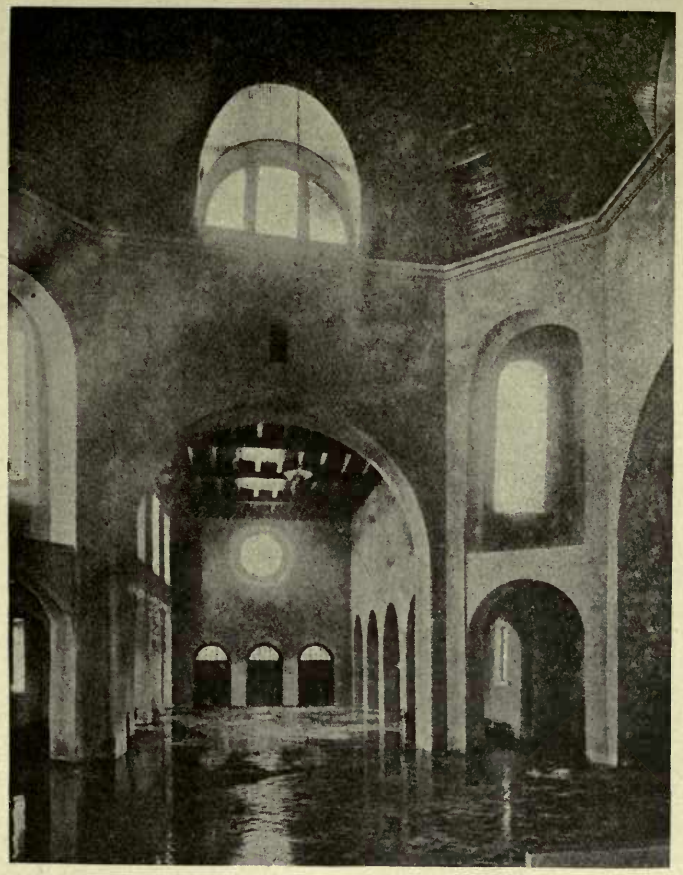


Fig. 541.—Interior View of Manila Cathedral (looking West)

Fig. 542.—Interior View of Manila Cathedral (looking East)

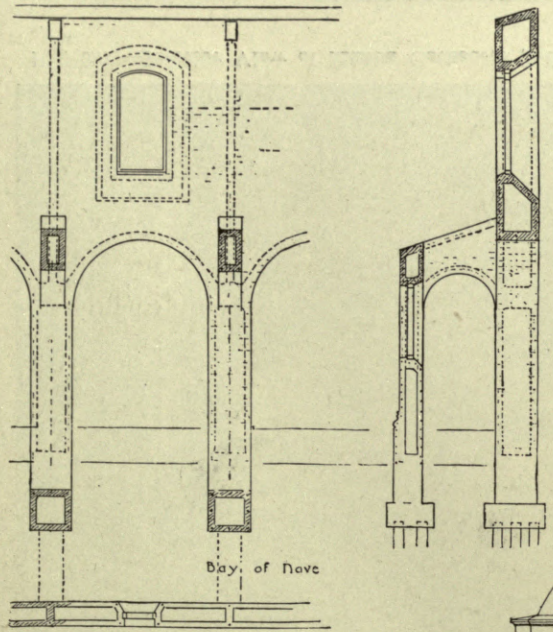
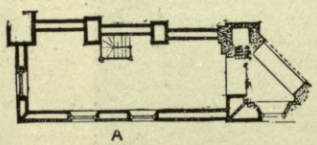


Fig. 544.—Details of Nave, Manila Cathedral



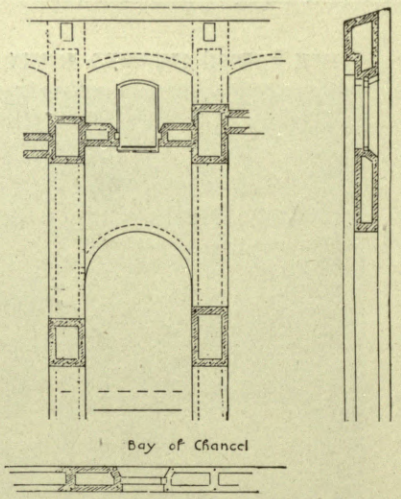


Fig. 545.—Details of Chancel

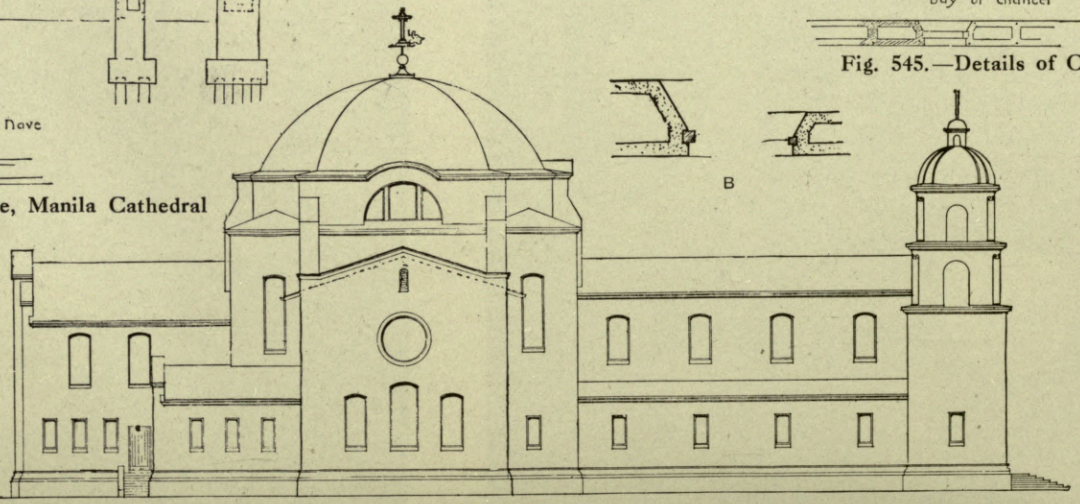


Fig. 543.—North Elevation of Manila Cathedral, with Plan of Organ Room (A) and Section of Window Jambs (B)

architects being Messrs. Sturgis and Barton, of Boston, and the contractors the Atlantic Gulf and Pacific Company. The illustrations give a good idea of the construction. The cathedral, which seats 1,000 persons, is built entirely of reinforced concrete, the vertical walls being hollow. The reinforcement used was mild steel in the form of round rods.

Figs. 546 to 549 (originally due to *Le Beton Armé*) show a cathedral at Poti, Russia, erected in reinforced concrete. It was designed by Prof. Marfield, and the construction is in accordance with the Hennebique system. The walls, though shown as solid, are built hollow, there being an outer wall 10 cm. thick, an air space of 18 cm., and an inner wall 8 cm. thick; the pillars also are hollow, the spaces being utilised in the ventilating and heating systems. The principal arches have a span of 12.30 metres, and the vertical distance from the springing to the topmost point of the grand cupola is 29.10 metres. Figs. 298 to 305 (pp. 194 to 196) show the erection of the dome.

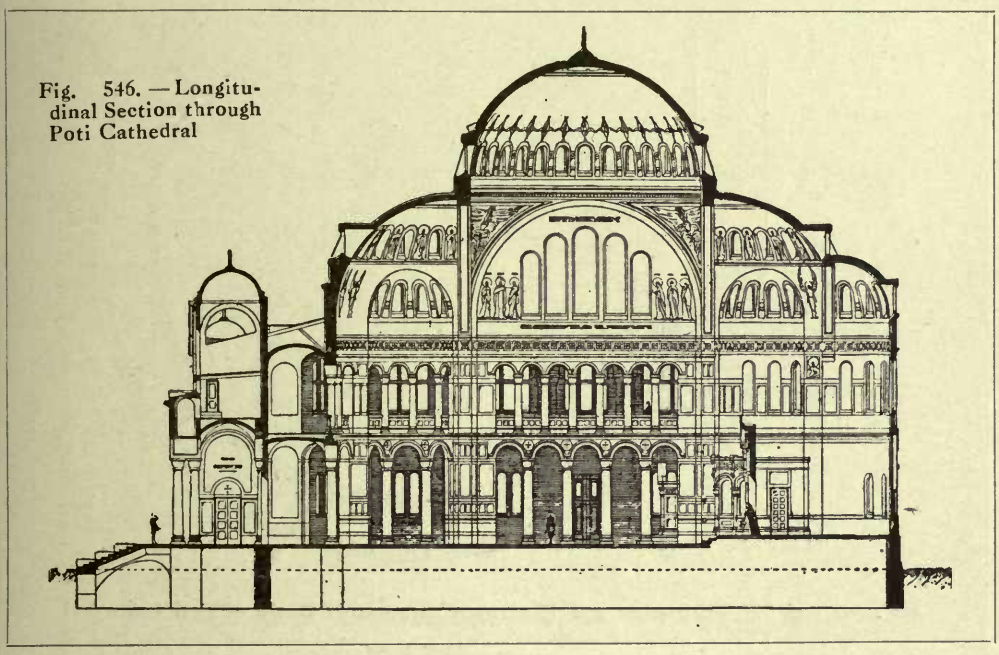
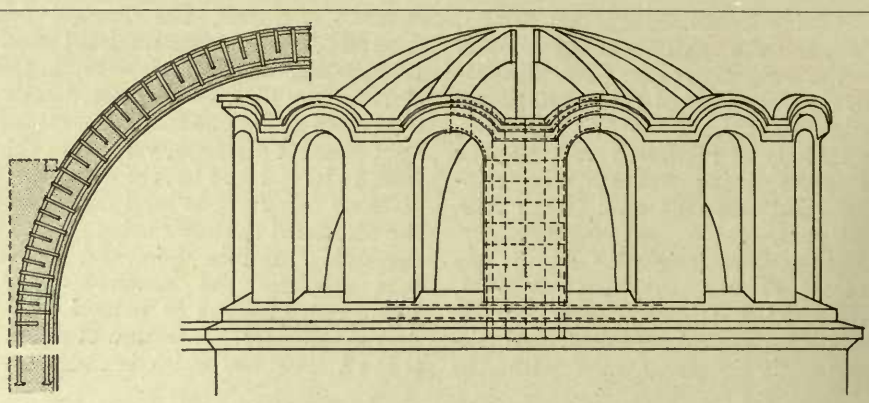


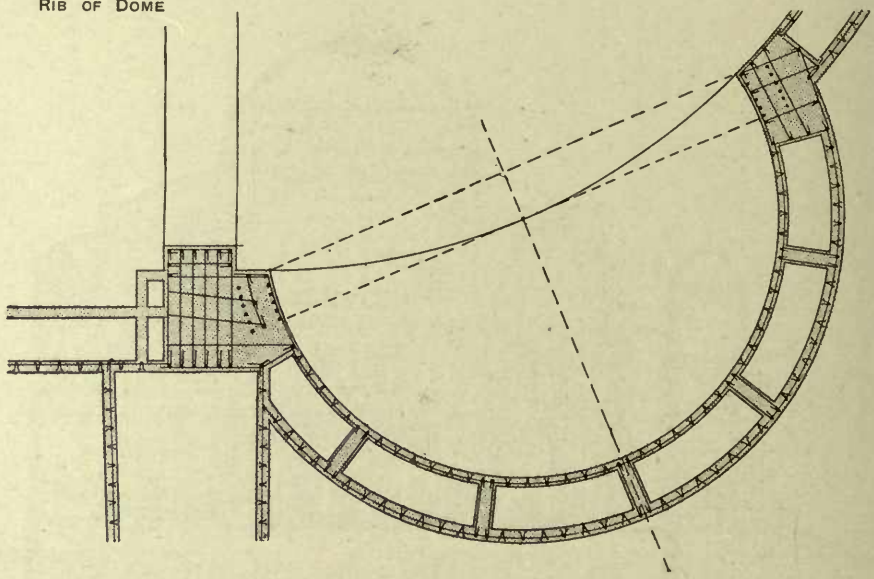
Fig. 546.—Longitudinal Section through Poti Cathedral

WESLEYAN HALL, WESTMINSTER

This building (architects, Messrs. Lanchester and Rickards, F.R.I.B.A.; general contractors, Messrs. Dove Bros.; reinforced concrete engineers, The Trussed Concrete Steel Co., Ltd.; constructional steelwork, Dawnay and Sons, Ltd.) is shown in plan and elevation by Figs. 550 and 551, details being illustrated by Figs. 552 to 565. It stands on a solid concrete platform, varying in thickness from 15 in. to 7 ft. 6 in., the under-side of the platform being at one level throughout, and the additional thickness being applied to the upper side. Half-in., ¾-in., and 1-in. trussed bars reinforce the concrete foundation, where extra loading or reduction in thickness necessitates. A feature of the building is the large dome which is carried on steel lattice stanchions, which are borne by grillage foundations, each pair of stanchions resting on eight 10-in. by 5-in. rolled steel joists, which are at right angles to seven 14-in. by 6-in. joists about 16 ft. long underneath, the whole of the steelwork in the foundations being enveloped with concrete.

In the basement of the building there are two large tea-rooms, each about 65 ft. long by 41 ft. wide, with ceilings in the form of reinforced concrete domes, carried in each case by six piers connected by curved reinforced concrete ribs. The elliptical dome in the centre of the ceiling springs from a rectangular plan, and the rise is only about 6 ft.; the concrete is 9 in. thick, reinforced with ¾-in. trussed bars at 12-in. centres.





Figr. 547 and 548.—Elevation and Horizontal Section of Dome of Poti Cathedral

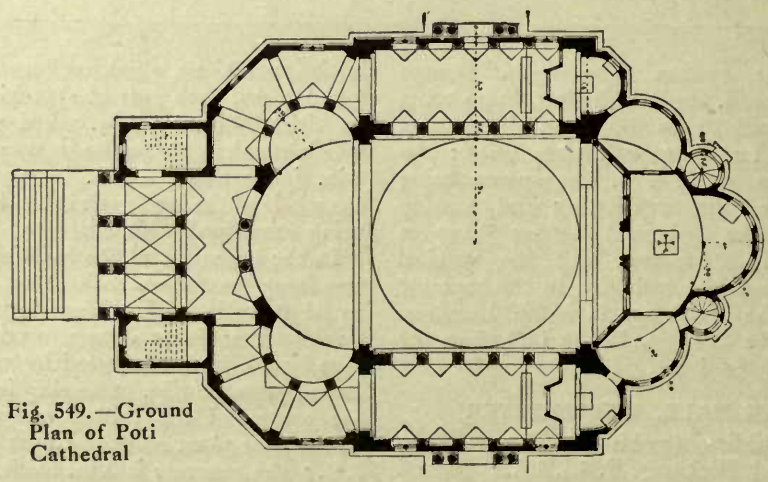
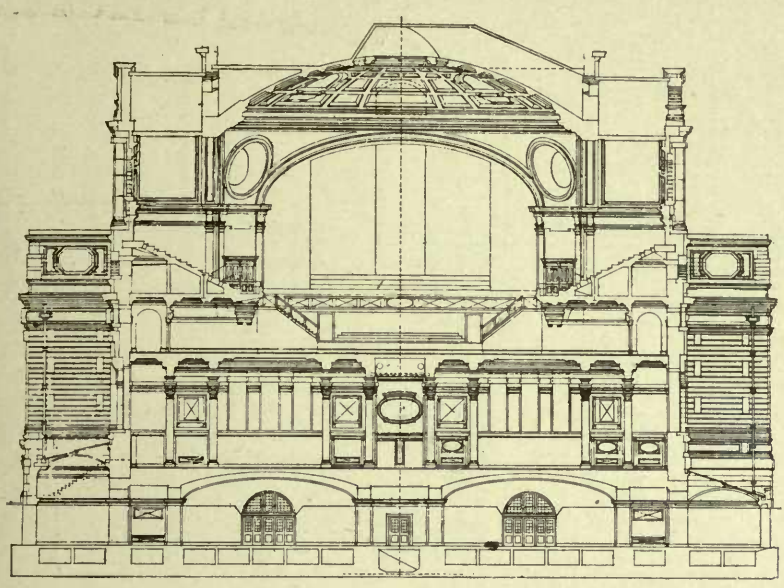
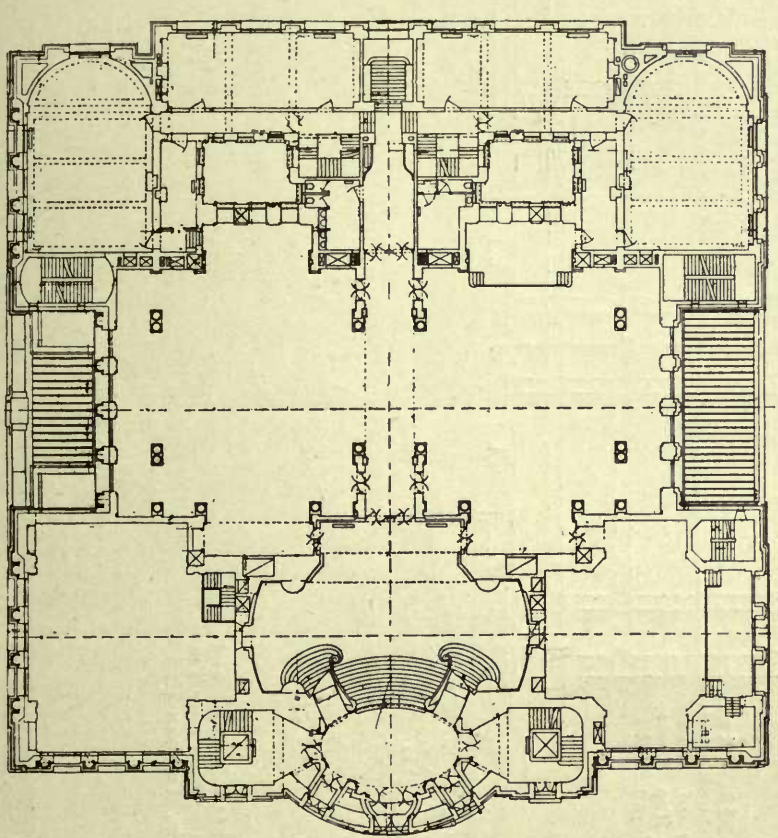


Fig. 549.—Ground Plan of Poti Cathedral





Figs. 550 and 551.—Cross Section and Plan of Wesleyan Hall

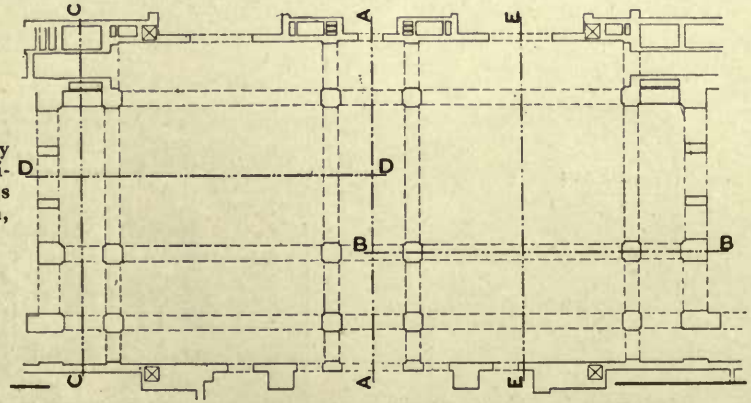


Fig. 552.—Key Plan showing Positions of Beams over Tea Room, Wesleyan Hall

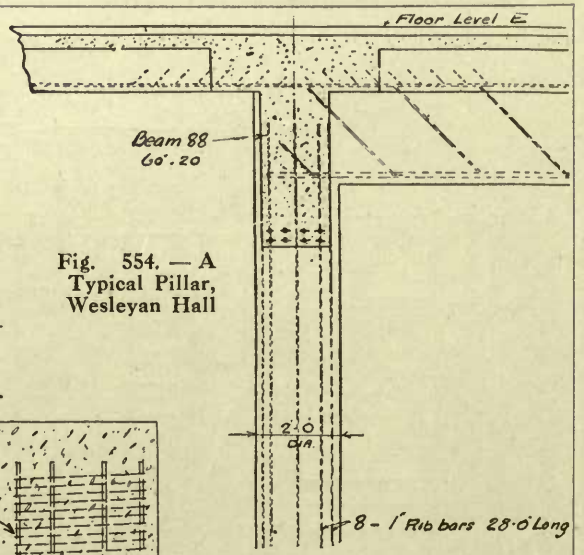


Fig. 554. — A Typical Pillar, Wesleyan Hall

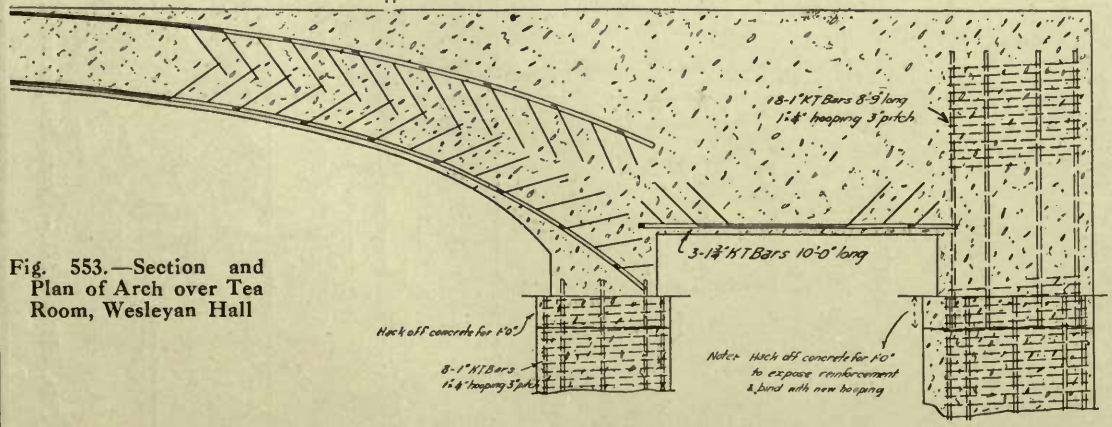
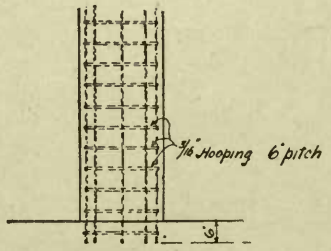
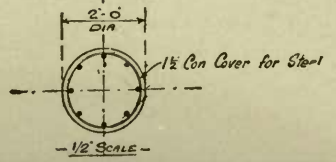


Fig. 553.—Section and Plan of Arch over Tea Room, Wesleyan Hall





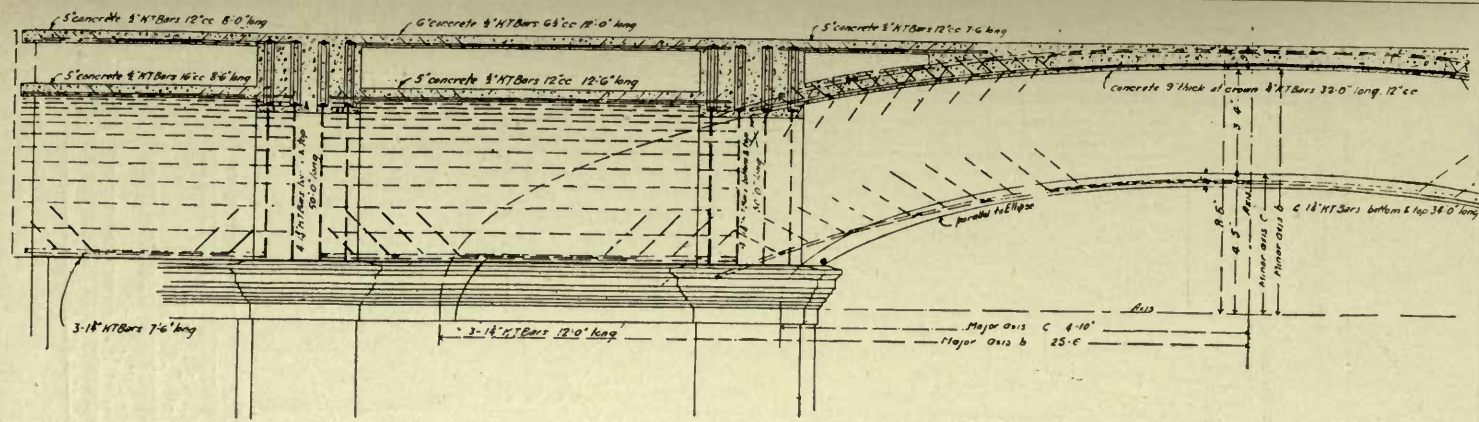


Fig. 555

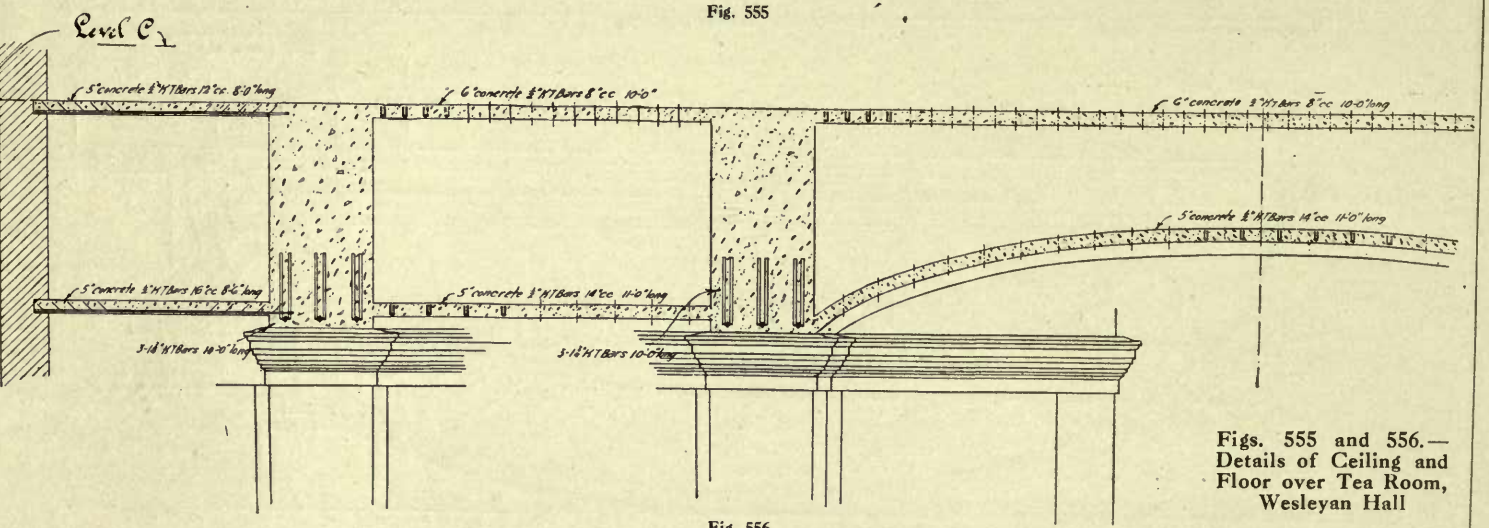


Fig. 556

Figs. 555 and 556.—Details of Ceiling and Floor over Tea Room, Wesleyan Hall

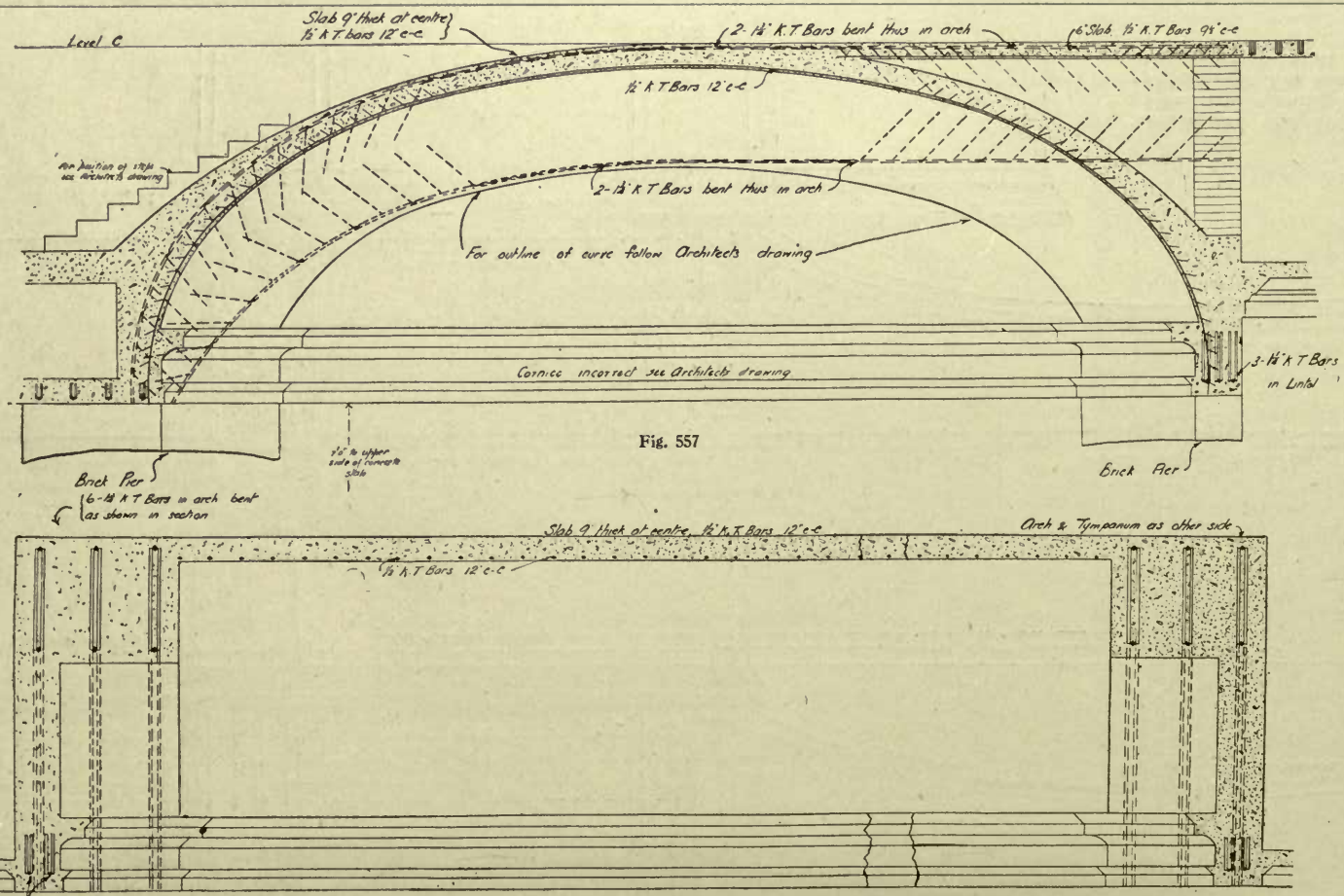


Fig. 557

Fig. 558

Figs. 557 and 558.—Details of Arched Ceiling to Basement Hall, Wesleyan Hall

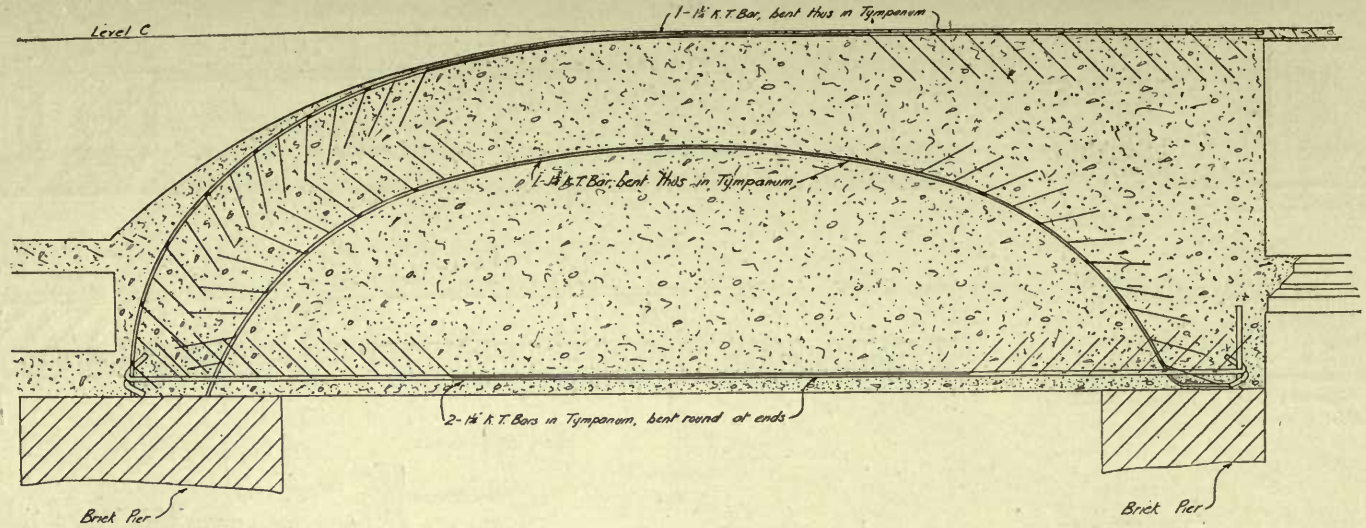


Fig. 559.—Cross Section through Part of Arched Ceiling to Basement Hall, Wesleyan Hall

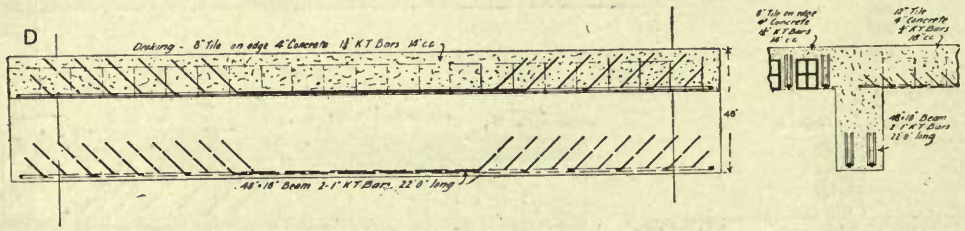
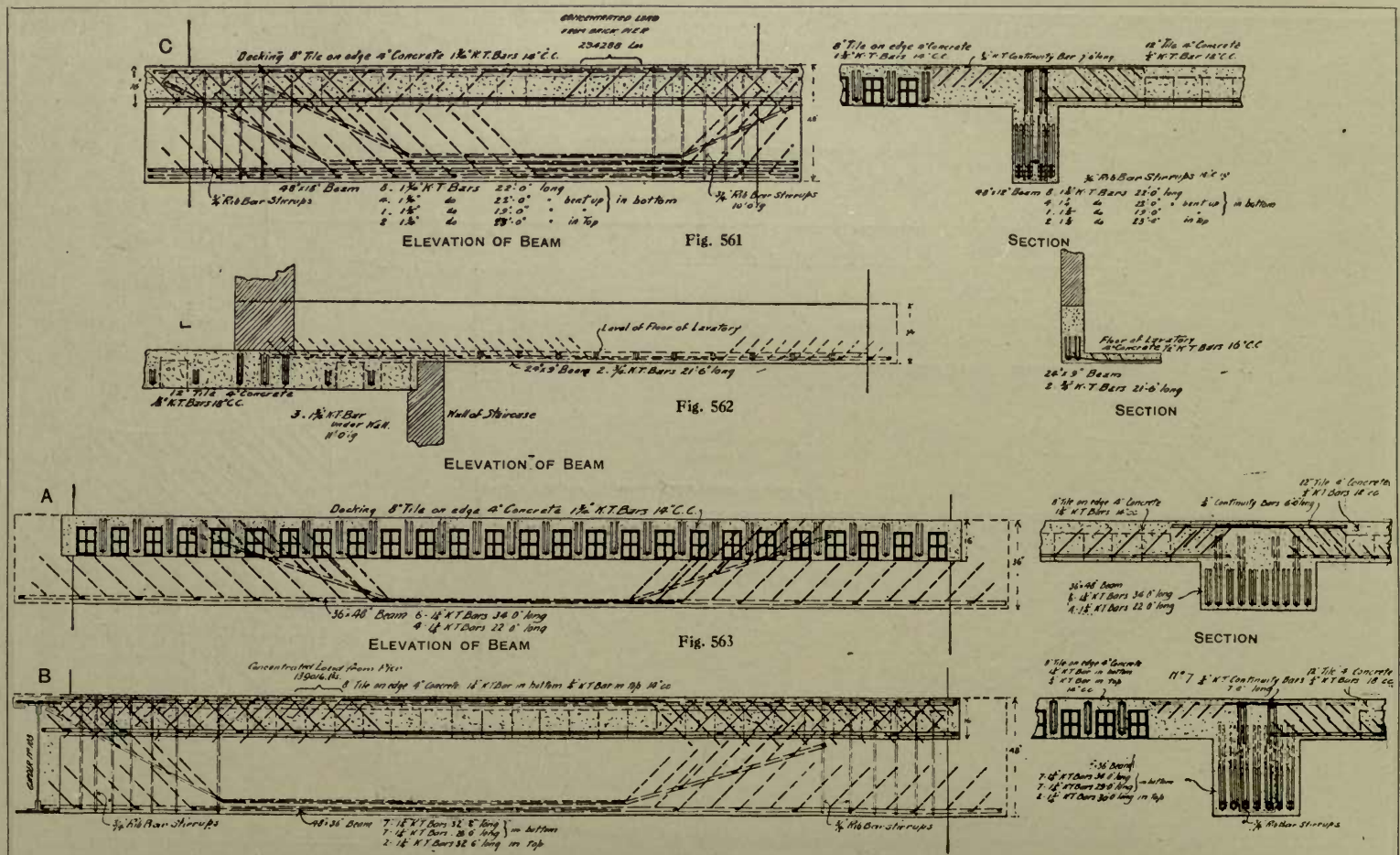


Fig. 560.—A Beam in the Floor of Conference Hall, Wesleyan Hall



Figs. 561 to 564.—Sections through Floor of Conference Hall, Wesleyan Hall

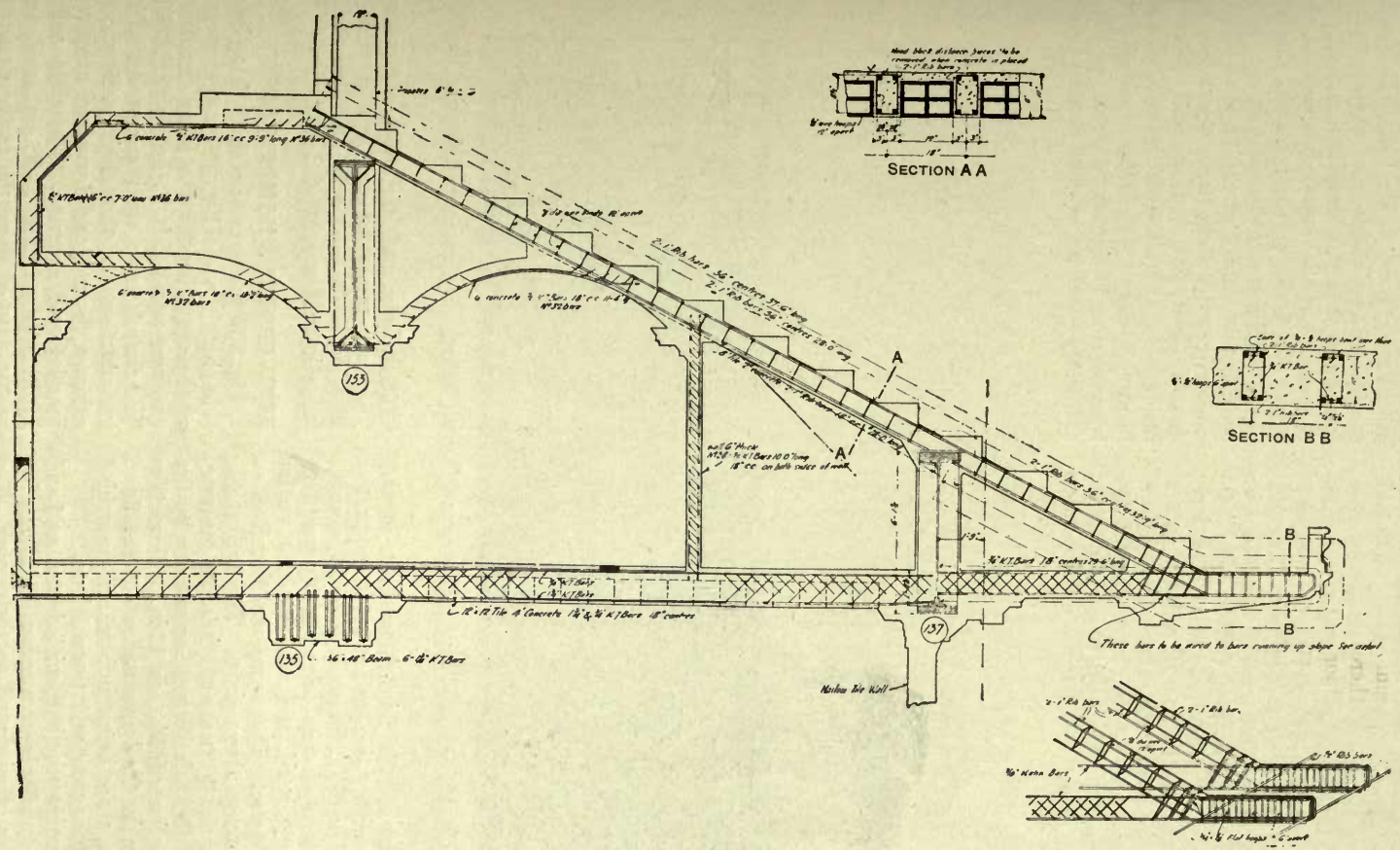


Fig. 565.—Section through and Details of East Gallery, Wesleyan Hall

Above the dome, as shown in the illustrations, comes the ground floor, consisting of a 5-in. thickness of concrete reinforced with $\frac{1}{2}$-in. bars at 12-in. centres, there being an open spandrel between this floor and the dome below. The piers which carry the weight of dome and floor are 4 ft. square and are reinforced by eight 1-in. trussed bars and 1-in. and $1\frac{1}{4}$-in. spiral hooping (pitch, 3 in.). On the next floor (the first floor) is a large conference hall, the floor of which is carried by reinforced concrete columns which have been illustrated in an earlier chapter (*see* pp. 177 to 180). Their height is about 28 ft., and their diameter 2 ft., the reinforcement consisting of eight 1-in. ribbed bars, hooped spirally with $\frac{3}{16}$-in. steel at a 6-in. pitch, the bars being $1\frac{1}{2}$ in. from the face of the work. The entasis of each column was formed in the mould itself, and was not built up by the plasterer. The forms are described at the reference above given. Throughout the building the floors are of reinforced concrete, lightened in a large number of cases by the introduction of 8-in. or 12-in. hollow tiles on edge in the lower surface of the floor, the 6-in. space between them being occupied by the concrete and trussed bars.

The conference hall, with domed roof and galleries, affords a number of interesting examples of reinforced concrete work. The galleries are of reinforced concrete, steel-plate girders being also used in places. The front of the gallery is carried by a reinforced cantilever, 14 ft. 6 in. long, the sloping portion of this making an angle of about 52 degrees with the horizontal and carrying the tiers for the seating accommodation. There are domes to the foyers in the front of the building, and there is a dome—a very conspicuous feature of the building—in the centre over the main hall. The latter is of double construction, that seen from the inside being of reinforced concrete, that seen from the outside consisting of a light steel framework. The concrete dome, of 114 ft. diameter, is carried on four concrete girders of 68 ft. span, and 18 ft. deep at the centre. They are supported on eight steel lattice stanchions, on each pair of which there is an estimated weight of 1,100 tons.

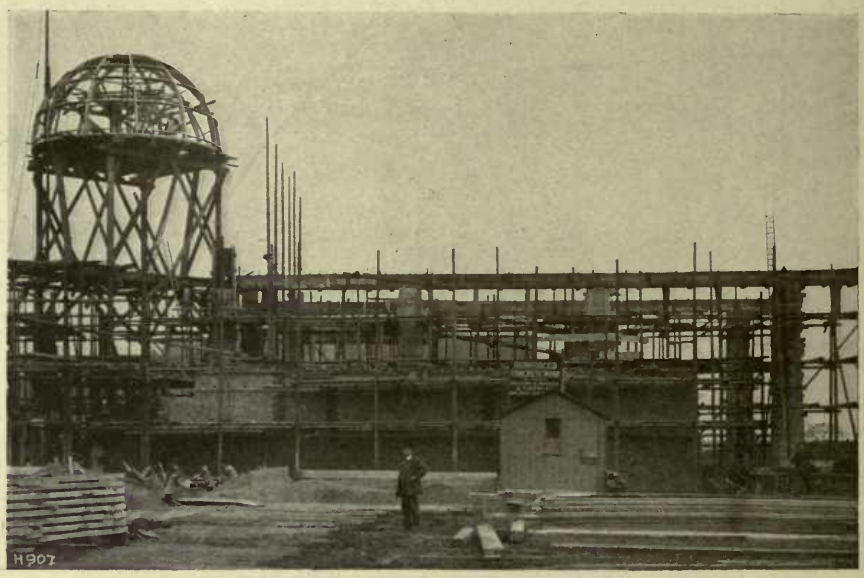


Fig. 566.—Free Church, Hampstead Garden Suburb, in Course of Construction

FREE CHURCH, HAMPSTEAD

Fig. 567 presents the chief elevations of this church, while Fig. 566 is a photograph of the centering for the inner dome. The architect was Mr. E. L. Lutyens, F.R.I.B.A. The church has a seating capacity of 1,100 in addition to the choir gallery. The columns, beams, and domes are constructed in reinforced concrete. The eight columns in the nave are about 28 ft. high, and, except in the case of those under the dome, they are 2 ft. 9 in. square for a distance of 9 ft.

from the floor, and circular above that height. They are reinforced by four $\frac{7}{8}$-in. square bars in the corners, and by eight similar bars uniformly spaced on the circumference of an internal circular cove. The eight bars reinforce the circular part of the column, the square bars bending into the circular portion. The vertical reinforcements are hooped with links of $\frac{1}{4}$-in. steel, spaced 12 in. apart. Each column rests on a reinforced concrete base measuring 8 ft. by 5 ft. The columns carry 18-in. square beams of 22-ft. span.

The columns that carry the dome are also about 28 ft. high, the first 9 ft. being 2 ft. 9 in. square, the upper portion being 2 ft. $4\frac{1}{2}$ in. square. The reinforcement is very similar to that of the round columns. These columns carry reinforced concrete arches, of which the rise is 13 ft., the depth at crown 4 ft. 6 in., and the thickness 2 ft. The arches themselves support the inner dome, having a radius of about 13 ft. and a rise of about 12 ft. Above the pendentives, which are 4 in. thick and reinforced with expanded metal, the angles are cut off by beams 4 ft. 6 in. deep and 2 ft. thick, which convert the plan to an octagon. There is a 6-ft. opening in the dome, the ring being reinforced with two $1\frac{1}{8}$-in. rods and two 1-in. rods. The main portion of the dome is reinforced with $\frac{5}{16}$-in. rods, circumferentially and radially. The outer dome has a timber framing which is supported by the frustum of a cone in reinforced concrete 30 ft. 6 in. high.

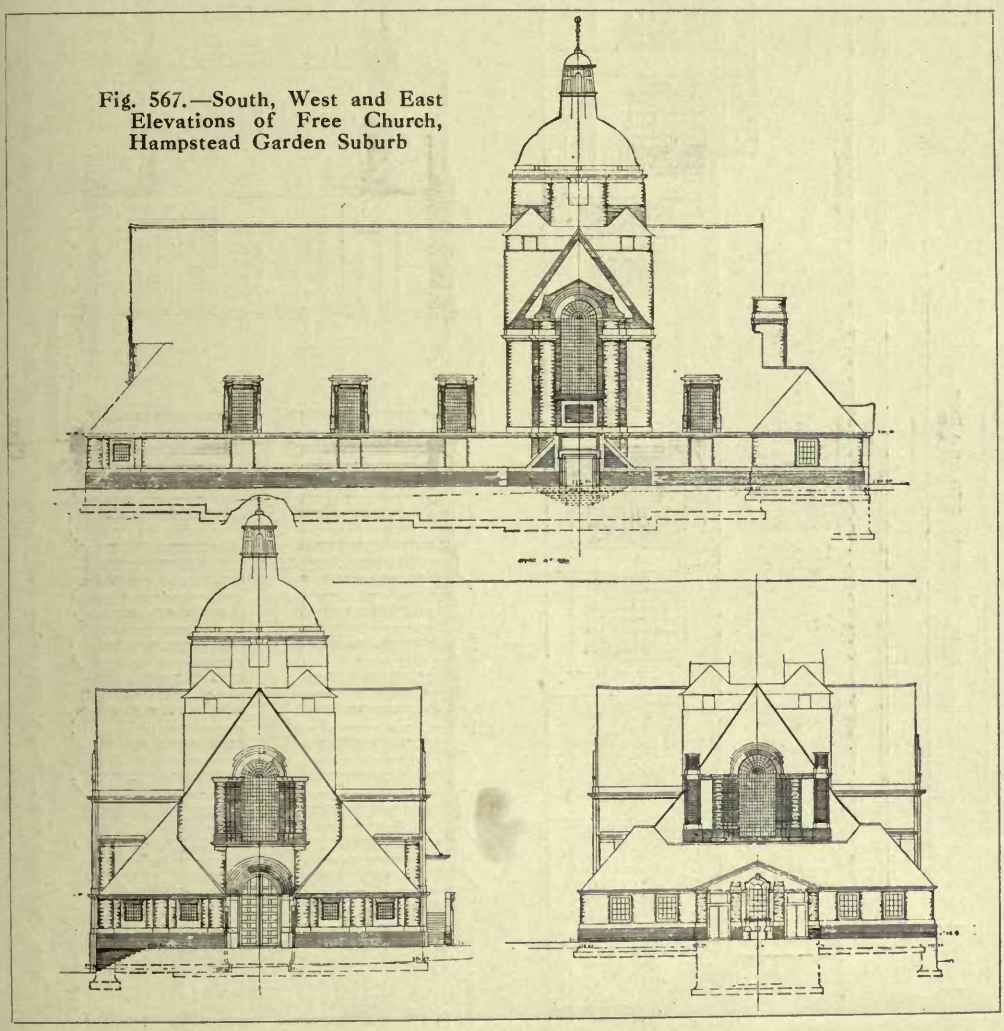
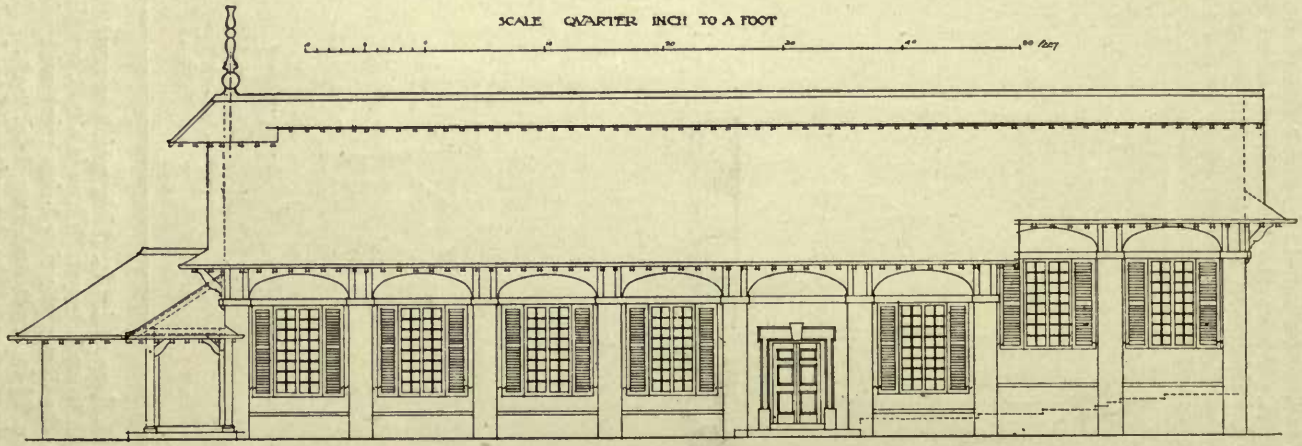
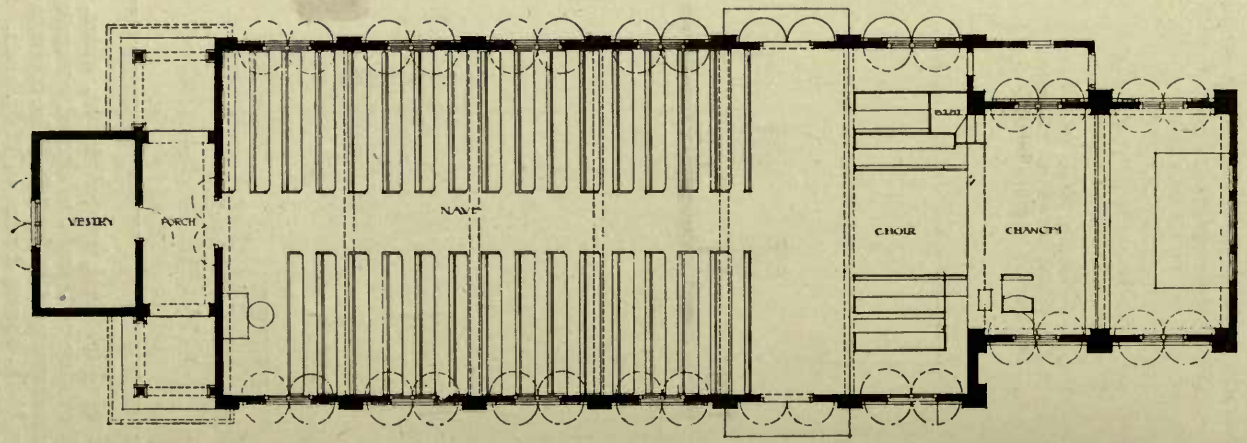


Fig. 567.—South, West and East Elevations of Free Church, Hampstead Garden Suburb





Figs. 568 and 569.—South Elevation and Ground Plan of St. Barnabas' Church, Jamaica

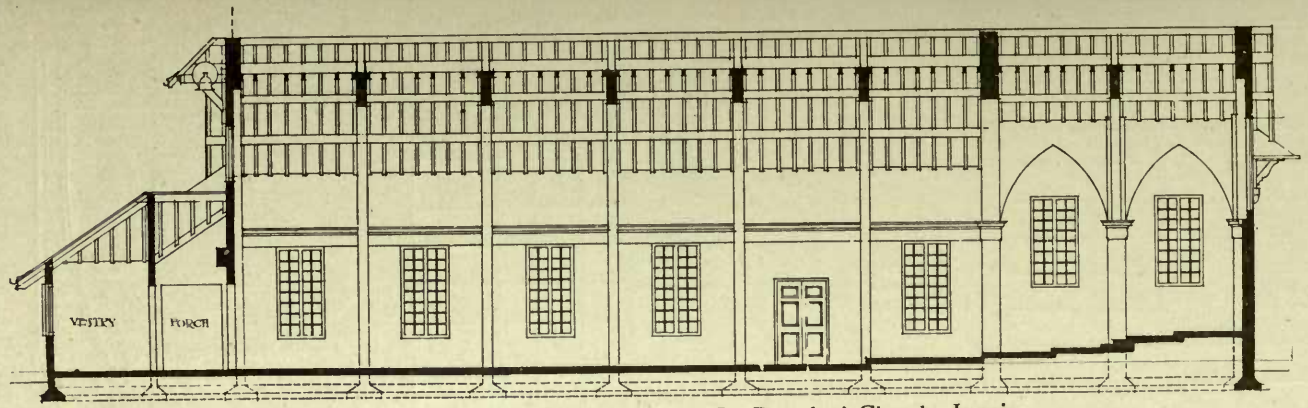
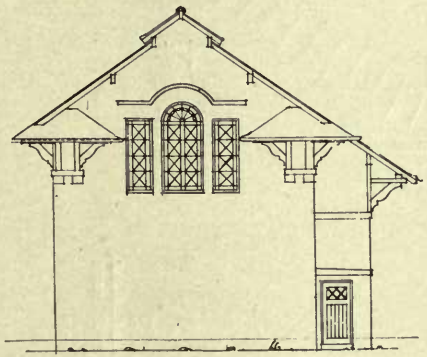
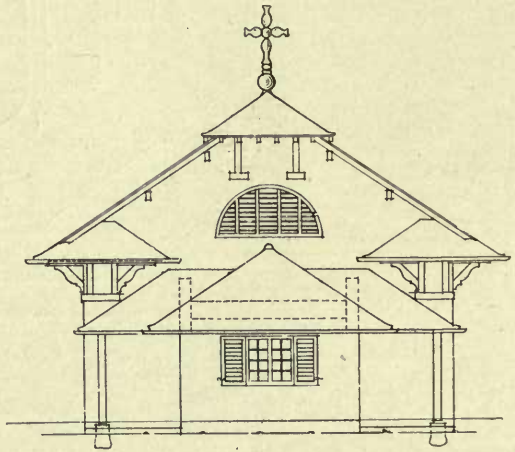


Fig. 570.—Longitudinal Section through St. Barnabas' Church, Jamaica





Figs. 571 and 572.—East and West Elevations of St. Barnabas' Church, Jamaica

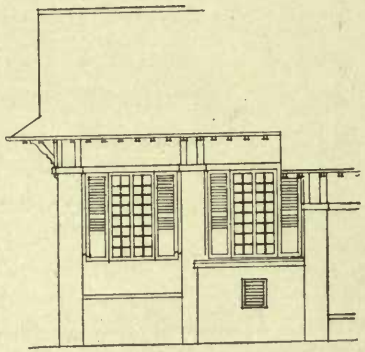


Fig. 573.—North Elevation of Chancel

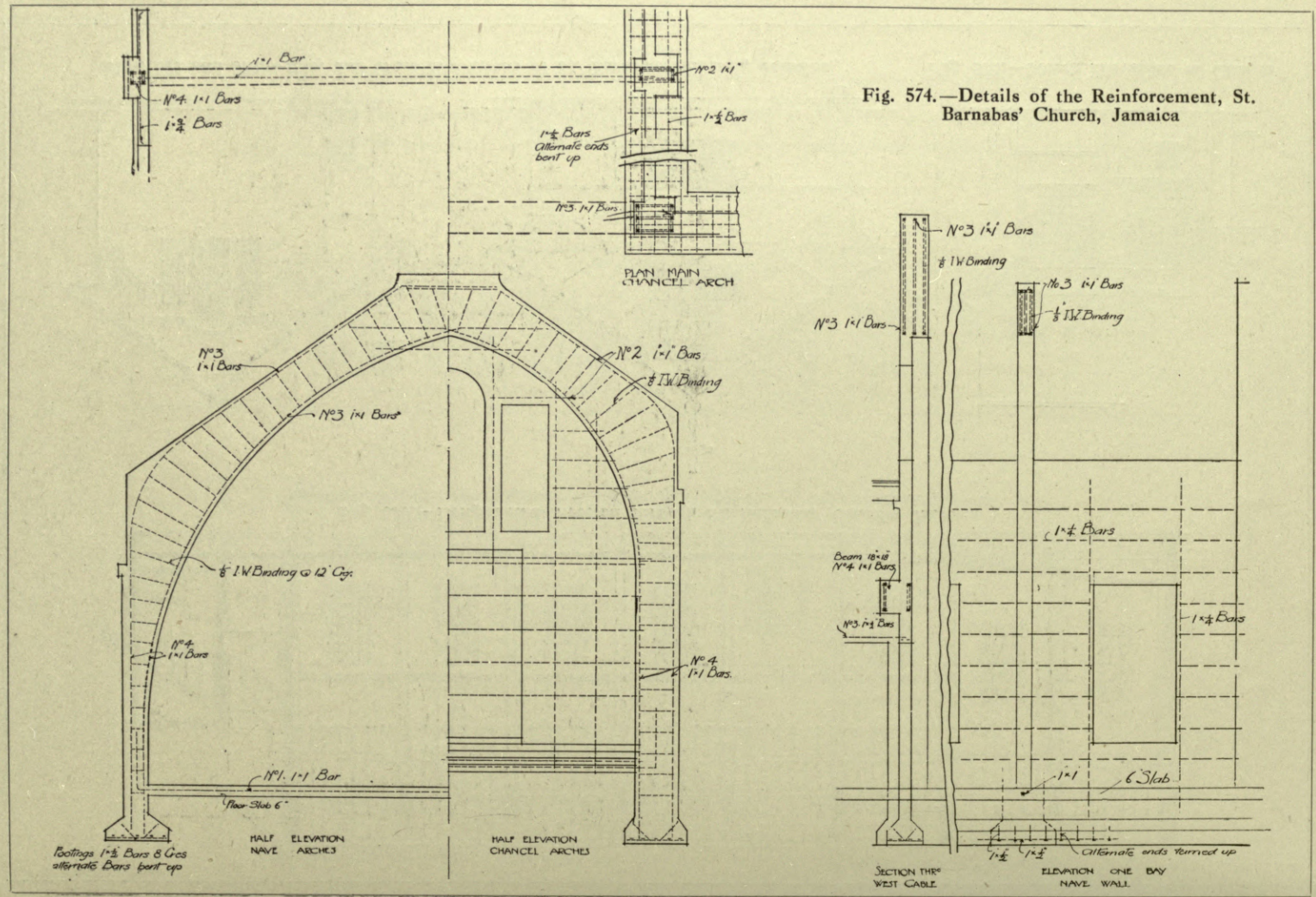


Fig. 574.—Details of the Reinforcement, St. Barnabas' Church, Jamaica

ST. BARNABAS' CHURCH, JAMAICA

Figs. 568 to 575 show a small church (St. Barnabas', George's Plain, Jamaica) erected to the designs of Mr. Hastwell Grayson, M.A., F.R.I.B.A., the reinforced work having been designed by Mr. Parr Emett. The church was planned for a coloured congregation of 230, including choir. George's Plain is close to, but not actually in, the districts liable to hurricanes and earthquakes. In the first instance, concrete was adopted for the walls only on account of the heavy charge for freight and cartage on other walling materials, but later it was found desirable to design the principals in the same material to provide greater stability. The spars and joinery are in hard wood, as soft wood is quickly destroyed by white ants. The donor was anxious that through currents of air should be encouraged, and that no inaccessible spaces should be left where scorpions might breed. The whole of the reinforcement consists of ordinary commercial bars, in rather long lengths, bound together with $\frac{1}{8}$-in. wire. The bars in the arches and across the floor for tying in the feet of the arches

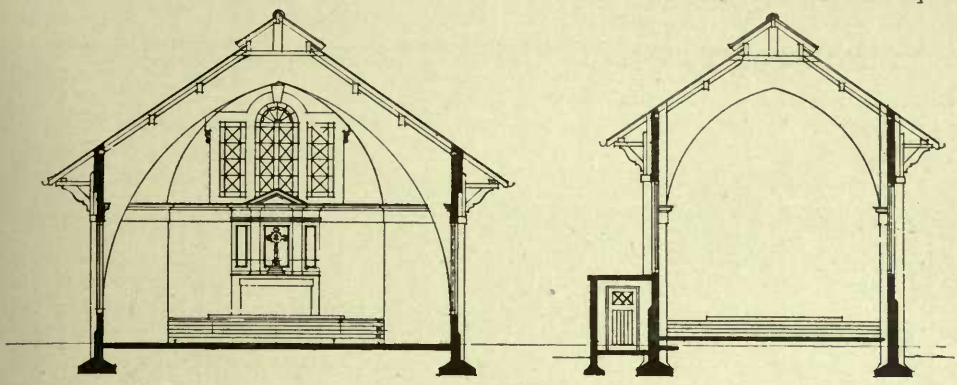


Fig. 575.—Sections through Nave and Chancel, St. Barnabas' Church, Jamaica

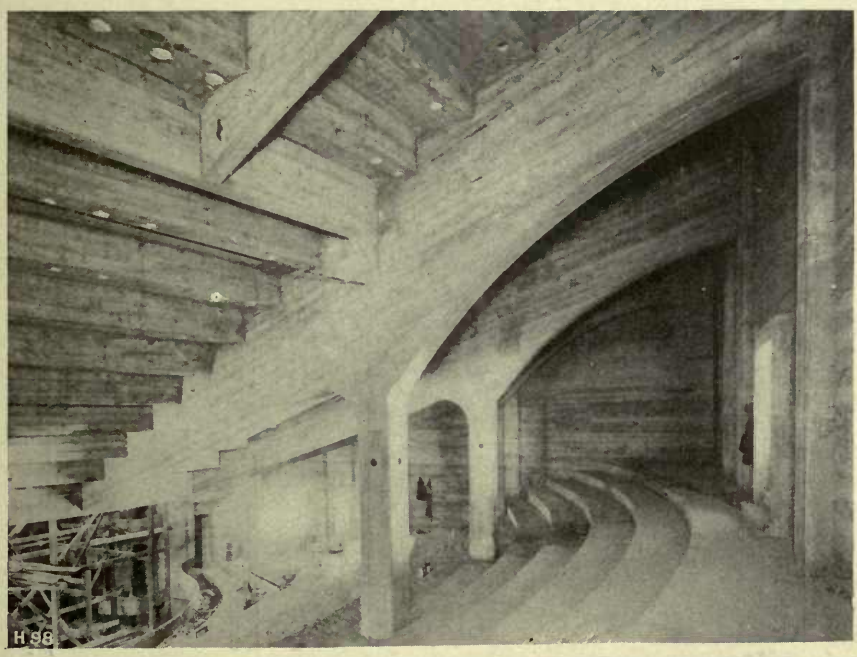


Fig. 576.—Balcony and Cantilevers in Majestic Theatre, Los Angeles

are 1 in. by 1 in. square, those in the foundations, 1 in. by ½ in., and those in the walls 1 in. by ¼ in.

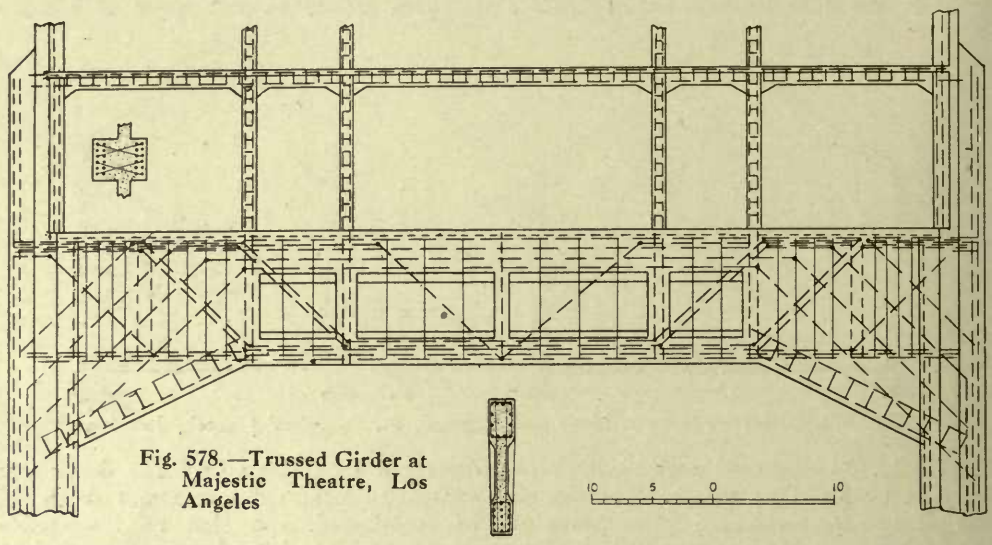


Fig. 578.—Trussed Girder at Majestic Theatre, Los Angeles

MAJESTIC THEATRE, LOS ANGELES

Whilst this theatre (*see* Figs. 576 to 580) is built of reinforced concrete entirely, only one or two of the most prominent features need be alluded to here. Fig. 576 is a view of the balcony and of the cantilevers which carry the gallery. The balcony, which has a free overhang of 30 ft., is carried by six cantilevers, the two side ones being brackets 18 ft. long and projecting from the truss columns on each side of the auditorium, whilst the four central ones (Fig. 577), having a clear projection of 29 ft. and 30 ft., rest on independent columns, extend back over the foyer, and are anchored in the wall at the rear of the auditorium. The cantilevers are 78 in. deep at their maximum section, the tensional reinforcement is seven 1⅜-in. square twisted bars, and the compression reinforcement three 1-in. square twisted bars, the two sets being connected by stirrups. To span the auditorium there are three trussed girders (*see* Figs. 578 to 580) having horizontal chords, and carrying a three-story building. Their length is 71 ft.; depth, 10 ft.; and widths at top and bottom 24 in. and 18 in. These trusses are reinforced at the top with twelve 1¾-in. square twisted bars, and at the

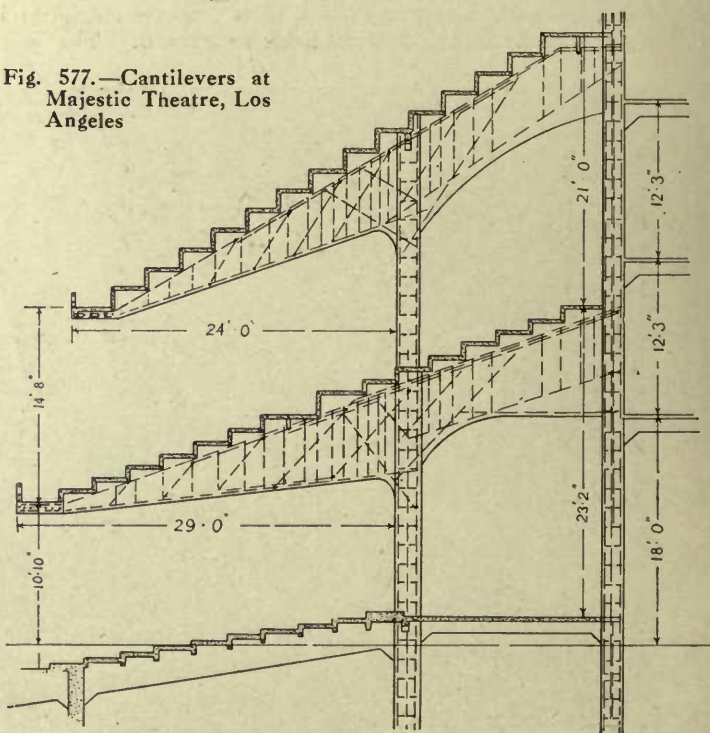


Fig. 577.—Cantilevers at Majestic Theatre, Los Angeles

bottom with fifteen $1\frac{5}{8}$-in. bars, and the bars are bent as shown in the diagram. It was thought desirable to splice the steel, the lap joints being 5 ft. to 6 ft. long, with three clamps each. The centering having been removed six weeks after pouring, each of the trusses carried a load of 500,000 lb. without measurable deflection, an additional 250,000 lb. on one of them causing a deflection of less than $\frac{1}{64}$ in. The diagram shows that the trusses are carried by columns 36 in. by 60 in. in section, the chief reinforcements of which are carried down into the footings. As the columns are more than 80 ft. long, the steel bars had to be jointed, laps of about 4 ft. being allowed, with three clamps to each. Only one bar in four has a joint at any one level. The columns, with the section slightly reduced, are continued up to the next floor level, where there is produced a thrust which is assumed to equal that caused by the maximum moment the elongation of the column is capable of carrying. The floor beams are built as horizontal columns to take the thrust; otherwise the large negative moment in the trusses at the support would have necessitated the employment of columns of immense sectional area.

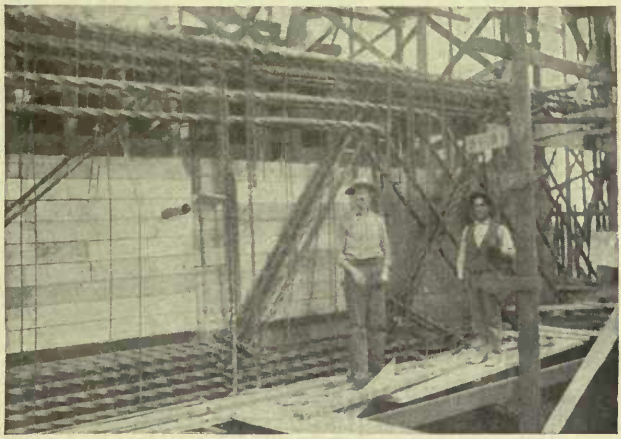


Fig. 579

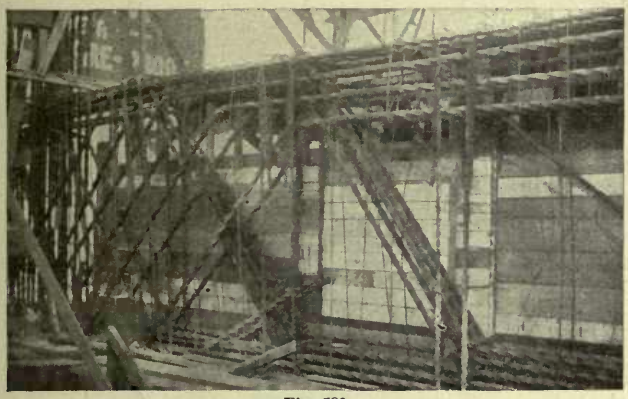


Fig. 580

Figs. 579 and 580.—Trussed Girder in Majestic Theatre, Los Angeles

NATIONAL GALLERY EXTENSION

The illustrations (Figs. 581 to 590) show the extension to the National Gallery, London, an extension that includes reinforced concrete columns, floors and roof constructed on the Kahn system. The floors and columns do not require description, but the roof and dome deserve attention. The roof principals, of the arched type, act as girders, tie-bars being thus unnecessary, as the beam takes up the whole of the stress due to the thrust of the arch. (The construction is on the same lines as that of the arched principal at the Hammersmith Baths, described on p. 198.) The dome has a span of 32 ft., and is 34 ft. 6 in. above the floor. The eight ribs are supported by four beams. At the top, the thrust of the ribs is taken by a reinforced ring round the turret; at the bottom, rods, which form a ring round the dome, take the thrust. This, with other details, is made clear in the illustrations. A slight detail, but one of practical importance, is the leaving of bolts in the concrete for the more convenient fixing of grounds for fibrous plaster, blind fittings, etc.

ALUMINIUM WORKS AT KINLOCHLEVEN

The works of the British Aluminium Company, at Kinlochleven, are not of reinforced concrete throughout, but a great many of the details—columns, bunkers, lintels, galleries, floors, etc.—are of that material, having been constructed on the Coignet system.

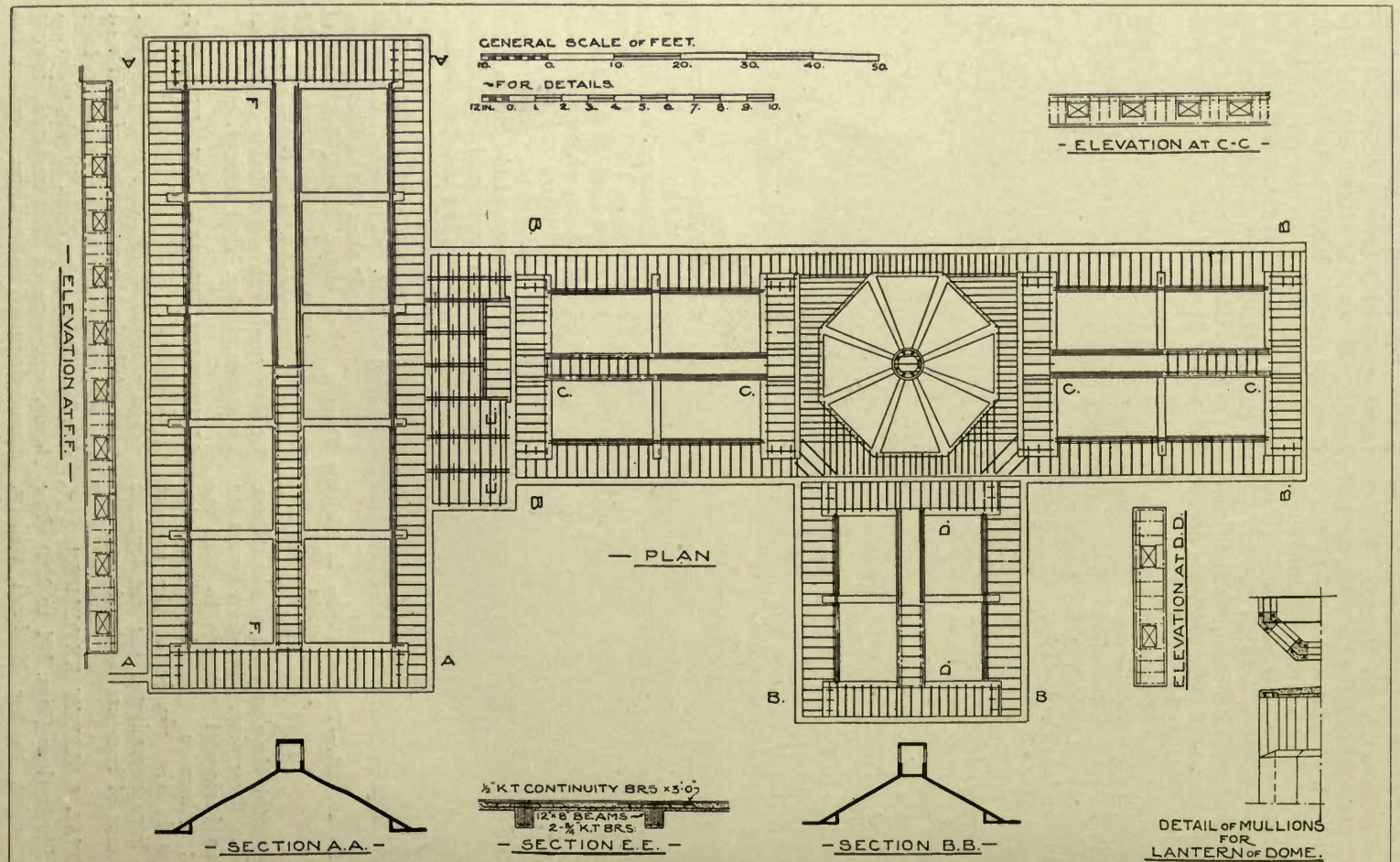
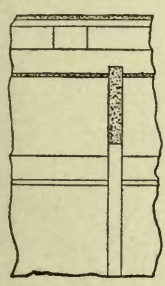
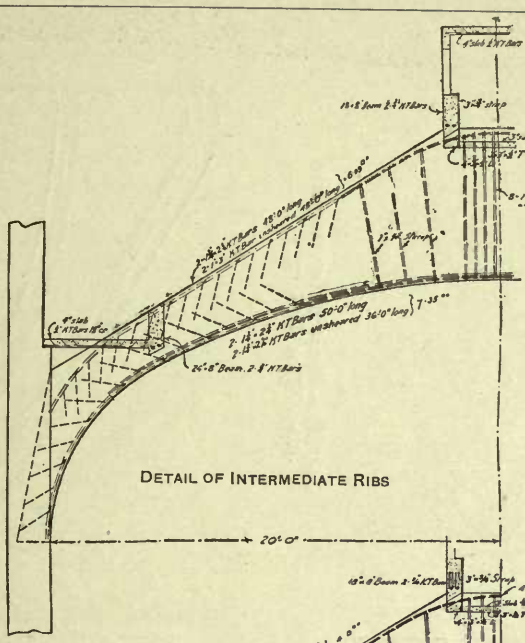


Fig. 581.—Roof Plan and Details, National Gallery Extension

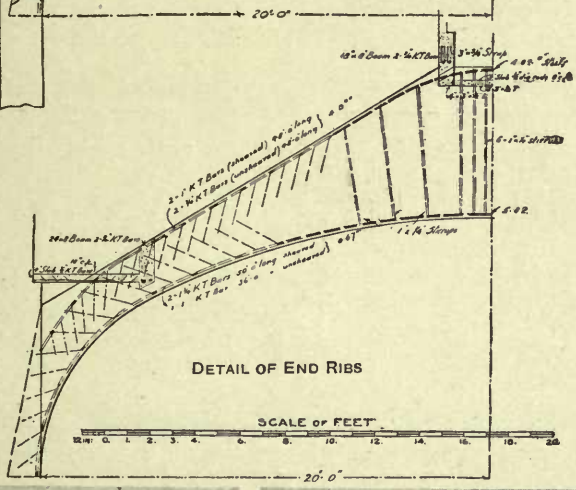


ENLARGED SECTION ON CENTRE LINE OF RIB



DETAIL OF INTERMEDIATE RIBS

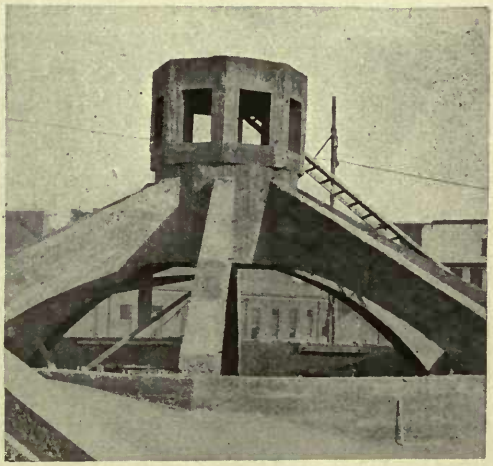
SECTION OF RIB



DETAIL OF END RIBS

SCALE OF FEET

Fig. 582.–Details of Intermediate and End Ribs of Dome Roof to National Gallery Extension



Figs. 583 and 584.—Dome Roof to National Gallery Extension

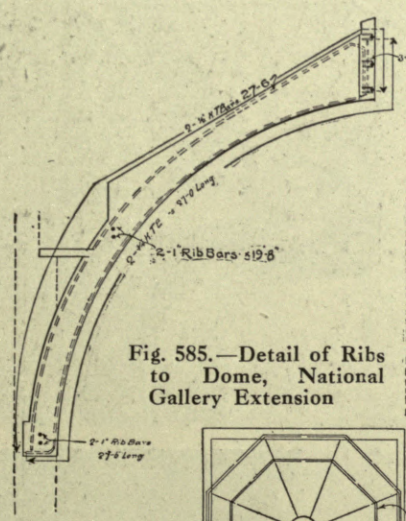


Fig. 585.—Detail of Ribs to Dome, National Gallery Extension

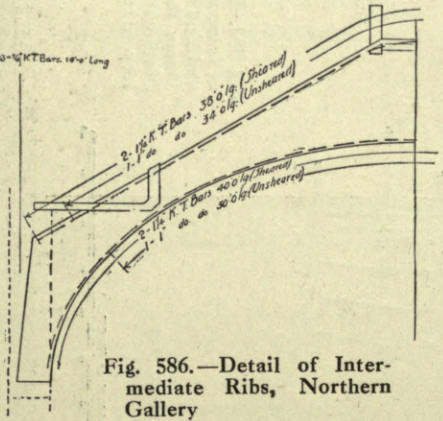


Fig. 586.—Detail of Intermediate Ribs, Northern Gallery

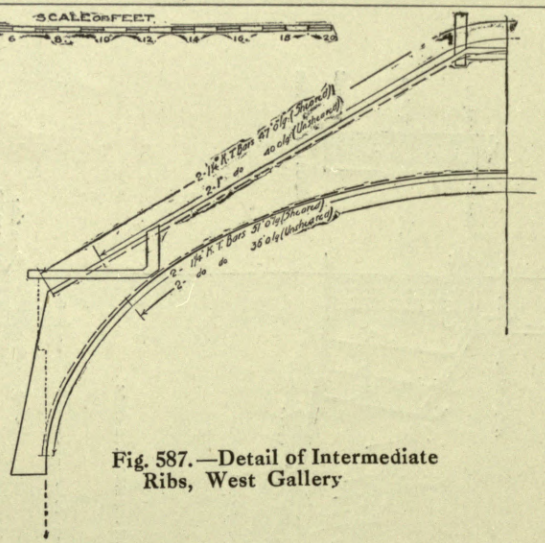
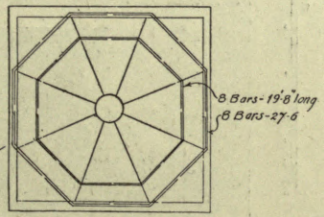


Fig. 587.—Detail of Intermediate Ribs, West Gallery



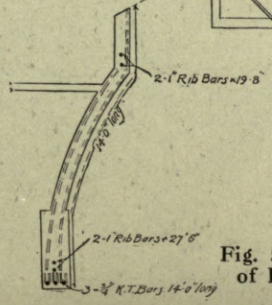


Fig. 588.—Detail of Pendentive

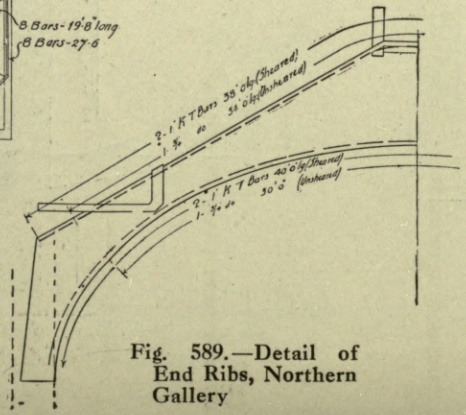


Fig. 589.—Detail of End Ribs, Northern Gallery

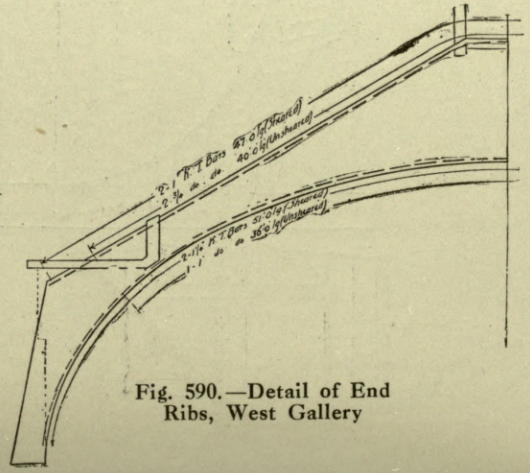


Fig. 590.—Detail of End Ribs, West Gallery

Probably owing to the unlimited supply of local rock which was crushed to form the aggregate, the estimate for bins, etc., in reinforced concrete was only ·55 that for the same work in steel. The bunkers are of large capacity, one of them having a

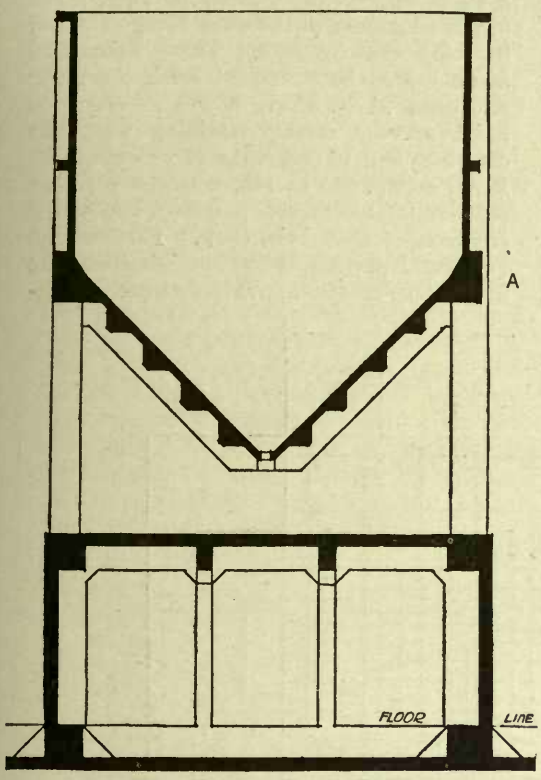


Fig. 592.—Section through Bunker at Kinlochleven Aluminium Works

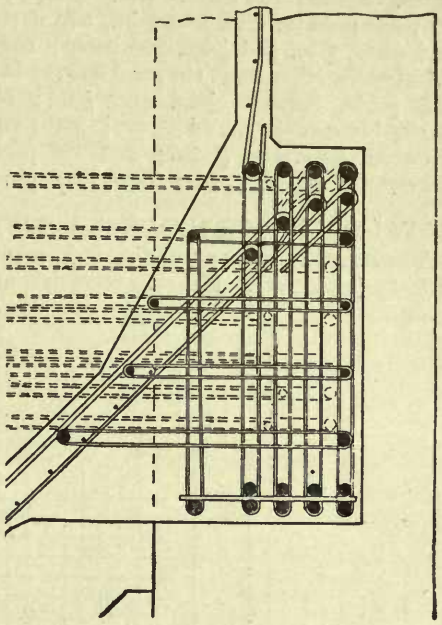


Fig. 593.—Detail of Beam of Bunker (A, Fig. 592)

cubic content of 80,000 cub. ft.; Figs. 591 to 593 show the construction of one of them, the beam at A (Fig. 592) being shown in detail by Fig. 593. The concrete was in the

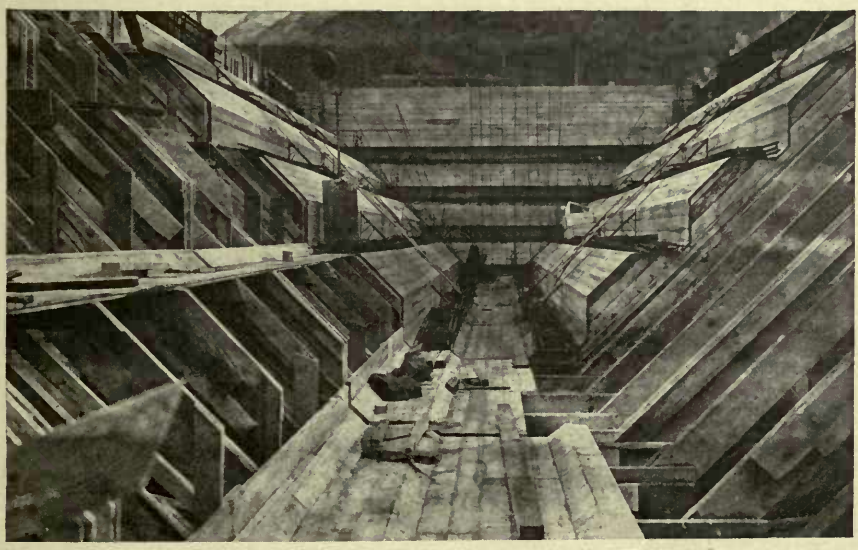
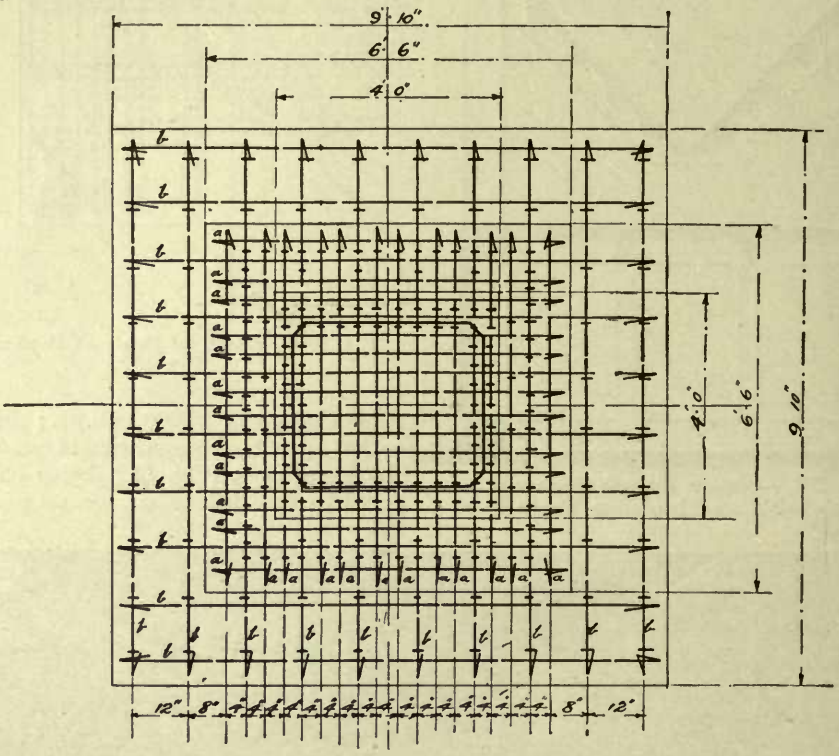


Fig. 591.—Bunker at Kinlochleven Aluminium Works

proportions of 3 : 5 : 10, and the aggregate was most carefully graded as follows: Passed $\frac{3}{4}$-in. mesh and retained on $\frac{5}{8}$-in. mesh, 80 parts; ditto, $\frac{5}{8}$-in. and $\frac{1}{2}$-in., 90 parts; ditto, $\frac{1}{2}$-in. and $\frac{3}{8}$-in., 570 parts; and ditto, $\frac{3}{8}$-in. and $\frac{1}{4}$-in., 260 parts; total, 1,000 parts. Similarly, the sand was graded: 250 parts, between $\frac{1}{4}$-in. and $\frac{1}{8}$-in.; 300 parts, between $\frac{1}{8}$-in. and $\frac{1}{16}$-in.; 200 parts, between $\frac{1}{16}$-in. and $\frac{1}{20}$-in.; and 250 parts, between $\frac{1}{20}$-in. and $\frac{1}{30}$-in.; total, 1,000 parts.

ROYAL INSURANCE BUILDING, LONDON

The Royal Insurance Building in Lombard Street (architects, Messrs. Gordon and Gunter; reinforced concrete engineers, The Considère Construction Co., Ltd.; contractors, Trollope and Colls) is constructed in steel as regards the general framing, much of the remainder of the work being executed in reinforced concrete. There being two floors below the pavement level, the lower one being 21 ft. down, it was necessary to build a very strong retaining wall, the drawings and photographs of which will be found on pp. 143 to 145, where it is shown that reinforced concrete possessed undoubted advantages over brickwork. Part of the horizontal portion of the retaining wall, as illustrated at the above reference, is con-



Figs. 594 and 595.—Plan and Section of Foundation Slab, Royal Insurance Building

structed as a beam with the object of distributing the pressure from the stanchions which carry the main walls of the superstructure. The retaining wall is undoubtedly an extremely interesting piece of work from the point of view of design. Figs. 594 to 598 illustrate some of the reinforced concrete work in the building. The drawings of the foundation slab illustrate the large number of stirrups used, this being a strong point of Considère designs. Reinforced concrete foundations were adopted for the steel stanchions throughout. The largest foundation is nearly 10 ft. square at the bottom, the offsets reducing this to 4 ft. square at the top. This base is reinforced with forty-four bars near the lower surface, and with forty-eight bars elsewhere, as illustrated in Figs. 594 and 595. Illustrations below show the strong room situated in the sub-basement floor, and measuring 8 ft. 10 in. high in the interior, divided into two compartments by a 10-in. wall, in which there is a small doorway. The outer walls and roof of the strong room are 10 in. thick and reinforced with both horizontal and vertical bars, which are connected by stirrups 4 in. apart. Reinforced floors, lightened by means of tiles, are used, the thickness of 7 in. being made of $2\frac{1}{2}$ in. of concrete, and $4\frac{1}{2}$ in. of hollow tile ($4\frac{1}{2}$ in. by 9 in.), but on both sides of the steel joists that carry the sides of the bay the concrete was kept solid for 18 in., and into this the lower bars were anchored. Bars also pass over the tops of the steel joists. Over the Lombard Street entrance is a reinforced concrete dome of 12 ft. diameter, the concrete being 4 in. thick, and reinforced circumferentially by $\frac{5}{16}$-in. bars at 12-in. centres, whilst the radial reinforcement is $\frac{5}{16}$ in. in diameter and spaced at 6-in. centres. Arches 5 in. thick are formed on four sides, so as to adjust the dome to the square plan (*see* Figs. 597 and 598).

WALL WITH POST BUTTRESSES

Figs. 599 and 600 show walls that are not of the continuous plain wall type, but are

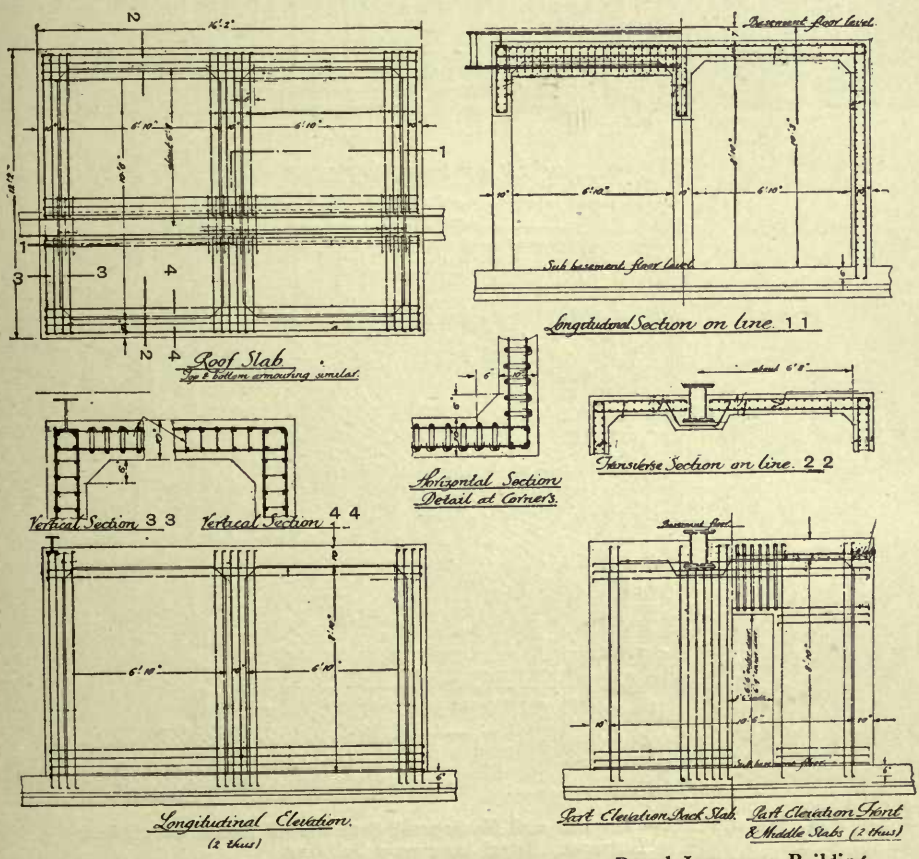
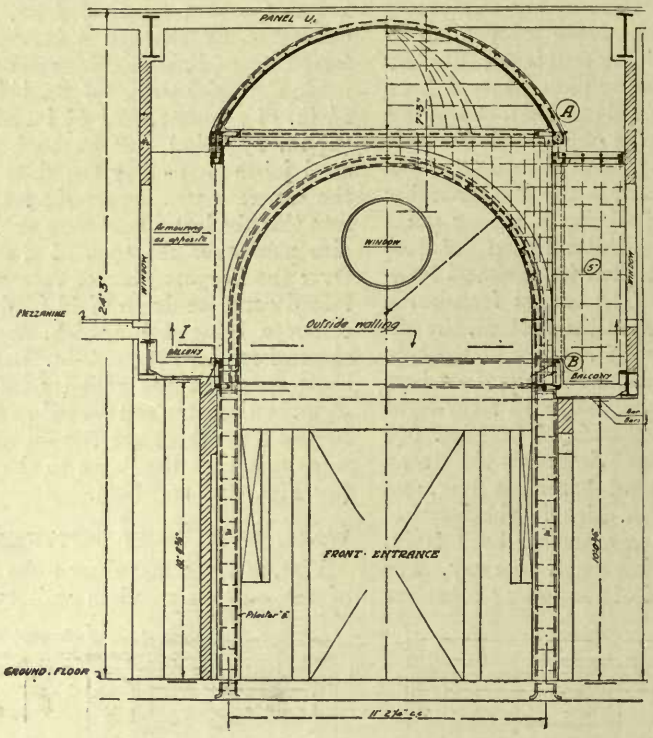
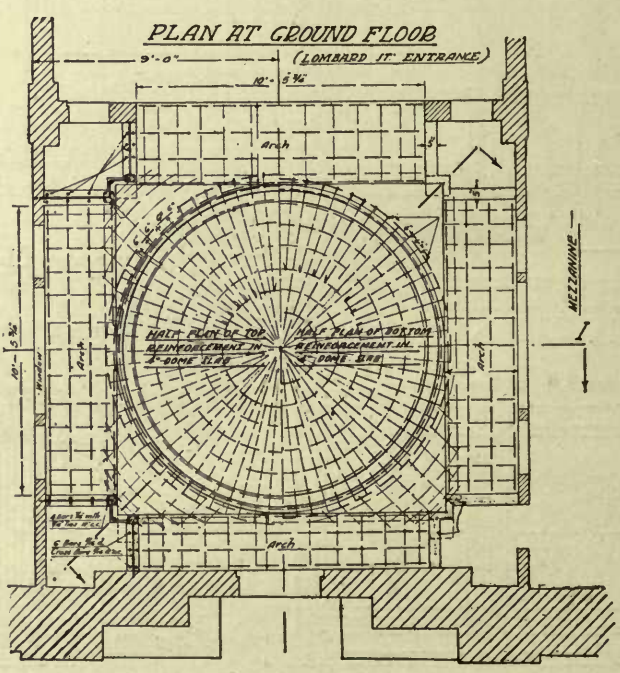
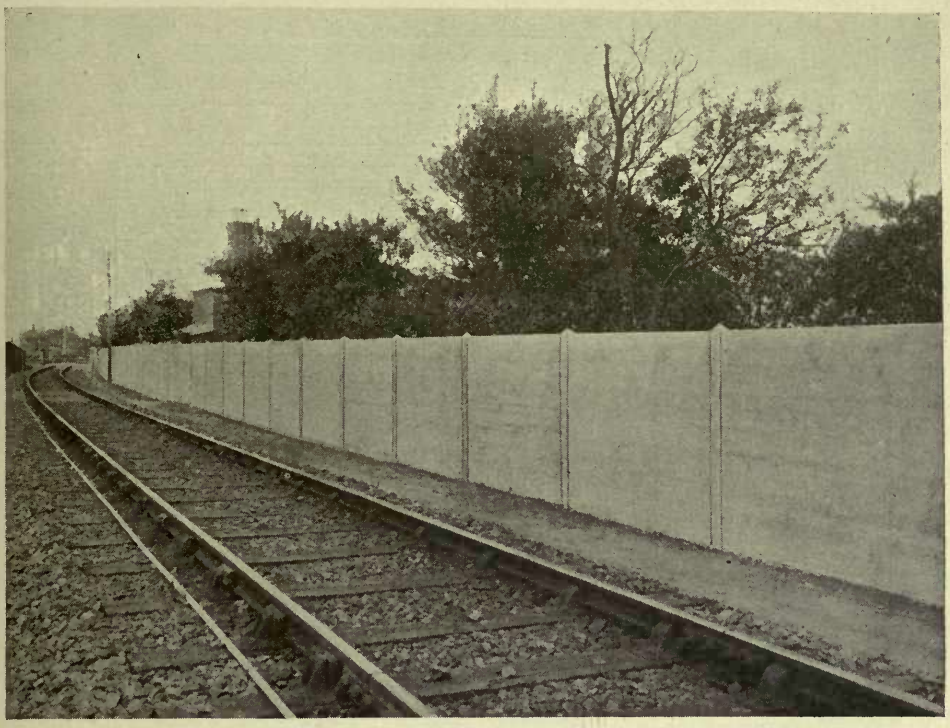


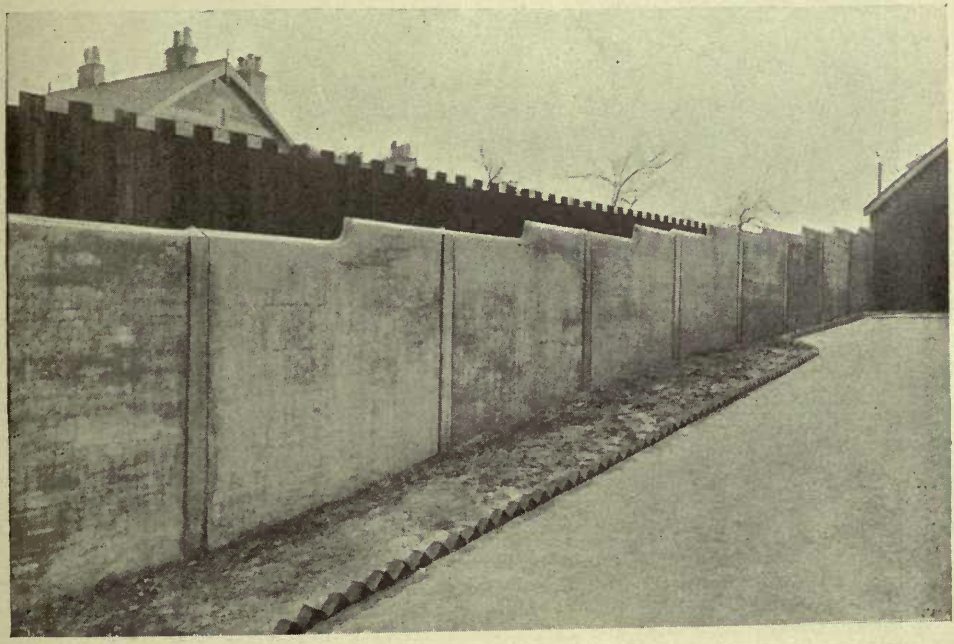
Fig. 596.—Details of Strong Room in Sub-basement, Royal Insurance Building





Figs. 597 and 598.—Vertical and Horizontal Sections of Dome over Main Entrance, Royal Insurance Building





Figs. 599 and 600.—Walls with Post Buttresses

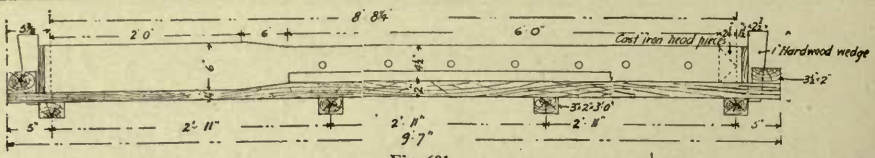


Fig. 601

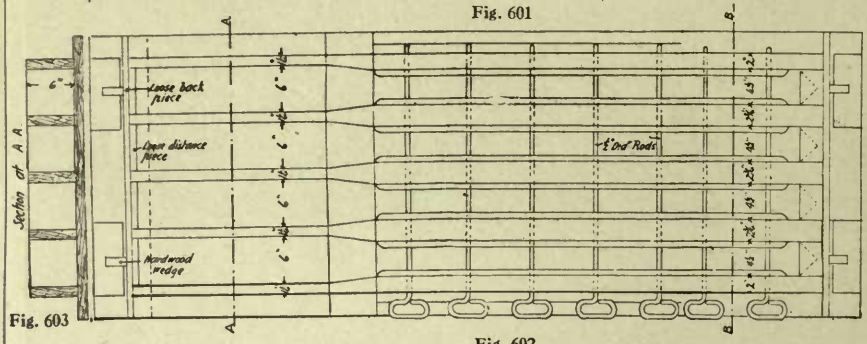


Fig. 603

Fig. 604

Fig. 602

Figs.—601 to 604.—Moulding Box for Fence Posts

Fig. 605.—Fence Posts

divided into bays by posts. These posts were manufactured at a depot in moulding boxes (*see* Figs. 601 to 604), and conveyed to the site. The design of the posts, as will be seen from Fig. 605, includes a groove on two sides; these are the sides into which the wall keys. Another and more particular object of the groove is that the contraction of the wall, if any, that may take place will be here, where it will not be seen, instead of cracking in any part of the wall, as might be the case in continuous concrete walls. A piece of wall of this description, after having been up for twelve months, showed no cracks in any part, but there was a slight contraction noticeable at some of the posts. The posts have several holes through them; these are for the horizontal reinforcing wires. The posts themselves have the usual reinforcement (*see* Fig. 606). The posts are sent to the site and erected just in the same way as ordinary fence posts.

The horizontal reinforcing wires, eight in number (providing for a wall 6 ft. high above ground line), are run through each post, and are strained tight with a special iron straining post, illustrated by Fig. 607, erected temporarily for that purpose. When this is done the result is a highly efficient strained-wire fence supported by concrete posts. Vertical rods are next interlaced with the horizontal wires; these have the effect of stiffening the wiring considerably. The shuttering may now be placed in position for moulding the wall. Ten bays of shuttering are necessary to keep a gang of five men going. Details of the shuttering are shown by Figs. 608 to 610, designed to fit up to the posts and leaving the desired thickness of the wall between them; the concrete is then put in in small quantities, and this, when well rammed with special spades, gives a face to the wall as smooth and hard as polished steel, no touching up being necessary. The richer the concrete the better the face; 4 to 1 gives a splendid result, in proportions of 5 of stone all passed through 1-in. sieve, 3 of sand, 2 of cement.

The shuttering must be made very strong as the concrete may be rammed with the special spades to such an extent as to bend or break almost anything. The wood becomes wet through, and in that condition it bends very easily; plenty of stiffening uprights are needed, as shown, with bolt-through, to keep the shuttering from spreading. In making the shuttering the boards, if dry, should not be cramped up tight, but should be just butted up to each other and the edges bevel faced, as otherwise, when they become wet and swell very much, they get badly out of shape; the bevel edge allows the boards to swell without buckling, and should they run over each other slightly they can be planed down. The boards for the shuttering should be selected from the best mernel or pitch pine, free from shakes, knots, and other imperfections, as the lightest imperfection on the surface of the boards spoils the face of the wall; bad defects in the face of the boards will bring the face of the concrete away with them. The holes for the bolts should all be burnt out so that the bolts may work freely in the holes when required to be removed. The

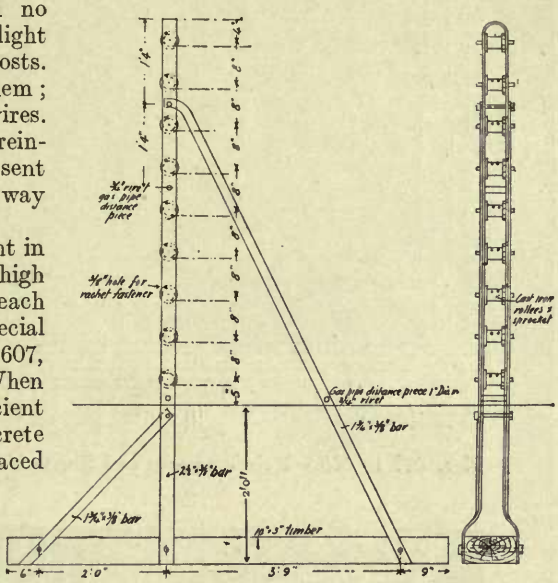


Fig. 607.—Iron Straining Post

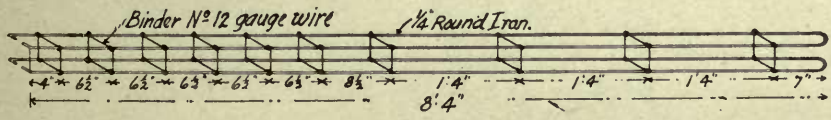


Fig. 606.—Reinforcement for Fence Posts

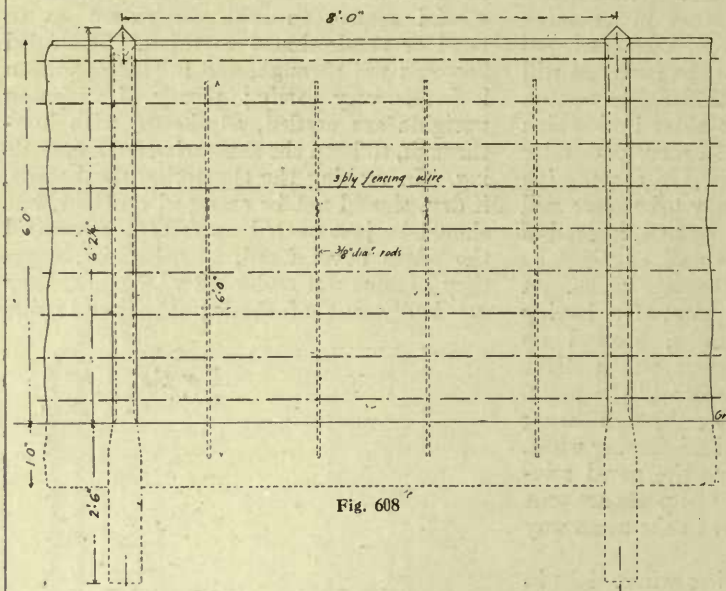


Fig. 608

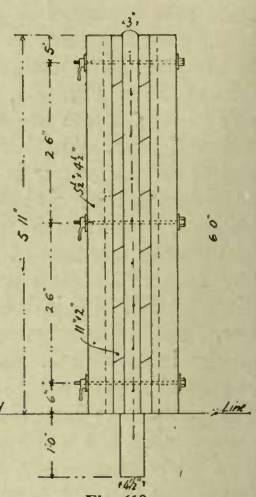


Fig. 610

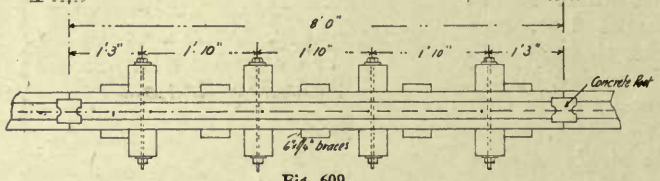


Fig. 609

Figs. 608 to 610.—Reinforcement and Shuttering to Wall

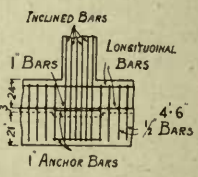


Fig. 612.—Reinforcement of Heel of Retaining Wall

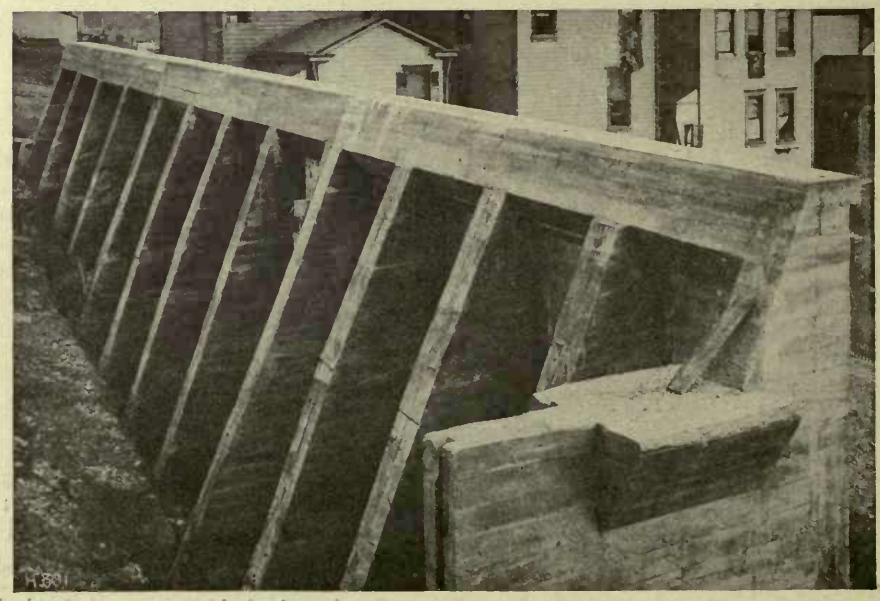


Fig. 611.—Retaining Wall, Dilworth Street, Pittsburg

bolts should have a square, quick-acting thread to enable the wing nut to be manipulated expeditiously. The washers for the bolts should be screwed on to the wooden uprights; they will then always be in the place where they are required.

The whole height of the shuttering has to be erected straight away on account of the bolts and upright timbers, as when these are once put in and screwed up no other boards can be put in. Therefore, with the full height of the shuttering erected, it is necessary to have long spades to ram the concrete at the bottom, shorter ones being used as the top is reached. With such a thin wall the space between the shuttering

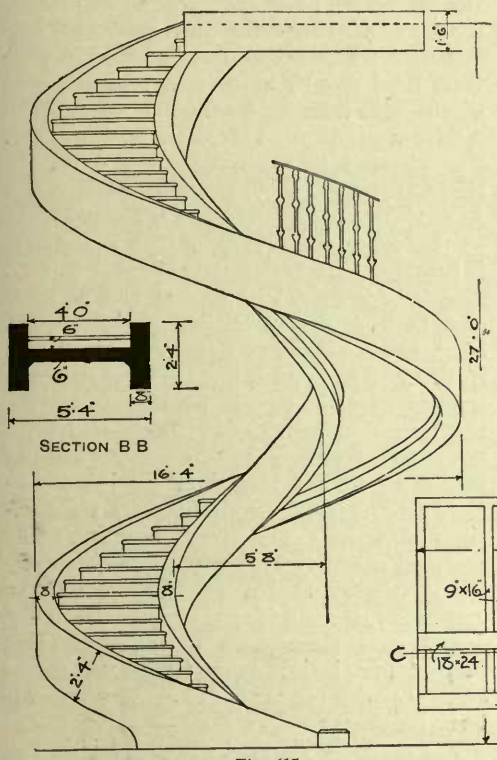
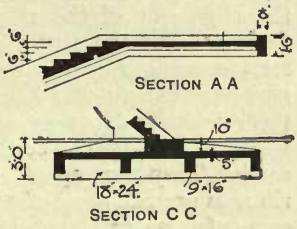


Fig. 615

Fig. 613



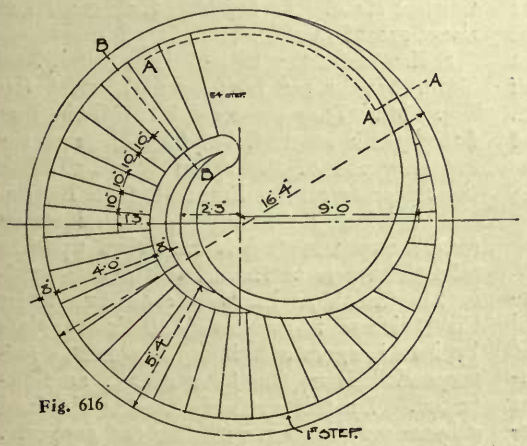


Fig. 616

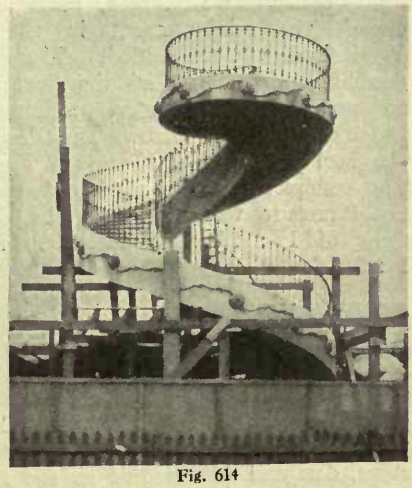


Fig. 614

Figs. 613 to 616.—Photographs, Elevation, Plan and Sections of Spiral Staircase

and the wire reinforcement is less than $1\frac{1}{2}$ in.; therefore a simple plain spade of hard-wood or iron, free from projections, must be used.

RETAINING WALL

Fig. 611 shows part of a retaining wall erected in connection with the construction of Dilworth Street, Pittsburg, in 1909. The wall is built on the site of a steep hill, upon which had been deposited clay and rubbish to a depth of from 8 ft. to 10 ft., and at a depth of $9\frac{1}{2}$ ft. below this shale of good quality was found. The reinforced wall was designed for three depths—35 ft., 30 ft., and 21·5 ft. respectively from top of wall to bottom of floor, and it rests upon piers extending along the front of the wall and underneath each rib. The required width of base, as computed, was 17·2 ft., and the width taken for the design was 17·5 ft., including a 6-in. toe projection. The front wall transmits the lateral thrust to the ribs, and is designed as a slab fixed at the ribs. At the ribs the reinforcing rods are placed through holes punched in angles made of structural steel, the object being to assure the alignment of the rods and to tie the front wall to the ribs. A double line of reinforcement is used in the front wall. The ribs were computed as cantilevers fixed at the base, where they are tied into the floor, which is held by the superimposed load of earth. They were spaced 9 ft. apart. The rib rods are run down the back of the wall, some of them extending into the projection at the heel, and some of them being placed under transverse 1-in. rods, which in turn engage the reinforcement in the floor. Vertical ties were added. There is a double reinforcement in the floor, so as to provide for negative moments at the base of the ribs. The design of the heel projection is shown in Fig. 612. The concrete in the reinforced parts of the structure was made in the proportions of 1 : $2\frac{1}{2}$: 5; elsewhere it was 1 : 3 : 7.

STAIRWAYS

The Franco-British Exhibition at Shepherd's Bush, London, W., in 1908, contained a few examples of reinforced concrete, one of the most interesting being a spiral stairway (*see* Figs. 613 and 614) of fifty-four steps, designed and erected on the Hennebique system. The line drawings (Figs. 615 and 616) show how graceful and striking the design was; and it will be understood, of course, that in practice such a stairway would be in monolithic connection with the floor beams, both at the top and the bottom. The stairway was exhibited as here illustrated, it standing on a relatively small area and supporting the spiral coils and the projecting platform solely by the elastic strength of the reinforced concrete construction. The height from ground to platform level was 27 ft., the over-all diameter of the spiral 16 ft. 4 in. The two curved stringers were 8 in. wide, by 2 ft. 4 in. deep. The distance between them was 4 ft., the risers were 6 in. deep, and the treads varied from $7\frac{1}{4}$ in. wide at the inner curve to $15\frac{3}{4}$ in. at the outer one. The cantilever platform is bounded by a kerb 8 in. wide by 1 ft. 6 in. deep, and finished by a 4-in. slab. The foundations were only 3 ft. deep below ground level, and consisted of a rectangular caisson, 16 ft. by 10 ft. in plan, formed of longitudinal and transverse beams connected at the top by a continuous slab. The calculations showed that the centre of gravity when the stairway was not loaded or carried a uniform load, passed through a point the position of which guaranteed stability. The result of calculations, assuming the stairway to be heavily loaded at one side of the centre line, showed that the centre of gravity would pass through a point which was only a few inches away from that corresponding with the centre of gravity for uniform loading.

The stairway shown in Fig. 617 is in the McIntyre building at Salt Lake City, U.S.A. The photograph shows the work in an unfinished state. As finished, the treads and risers are of marble.

Staircases at the Ritz-Carlton Hotel, New York, have some features of interest (*see* Fig. 618). They are straight until the first floor level is reached, but from there to the top they are of the spiral shape. Unlike the staircase at the Franco-British Exhibition previously referred to, the reinforced concrete construction is not relied upon to take the whole of the stresses due to loading, inasmuch as between each two floors the staircase is supported at three points by iron hangers cast into the concrete and suspended from the floor above, as illustrated. By this method there is obtained a beam construction with two end supports. The following particulars are due to

Engineering Record: The linings are on the cantilever principle. The forms for these staircases were quite a little problem in themselves. The soffit was moulded on a $\frac{3}{4}$-in. layer of 1 : 3 mortar, applied to $\frac{1}{4}$-in. wire mesh, which was attached to transverse pieces. This mortar was allowed to take a hard set, after which the longitudinal and transverse reinforcement was wired in, and the final cast of a complete staircase made at one operation. The soffit was plastered in the upper stories and basement, while in the cellar it was simply rubbed to give a finished appearance. No form was used for the treads, and the surface for the entire width of $3\frac{1}{2}$ ft. was hard trowelled. All risers and exposed strings were rubbed until smooth: the stairs were primed and enamelled. The lower and upper ends of each staircase are supported directly on the floor and beam construction of the building. All reinforcement consisted of square, cold-twisted steel bars, on which a unit tensile stress of 16,000 lb. per square inch was allowed. The longitudinal reinforcement consisted of $\frac{5}{16}$-in. square bars, spaced $2\frac{3}{4}$ in.

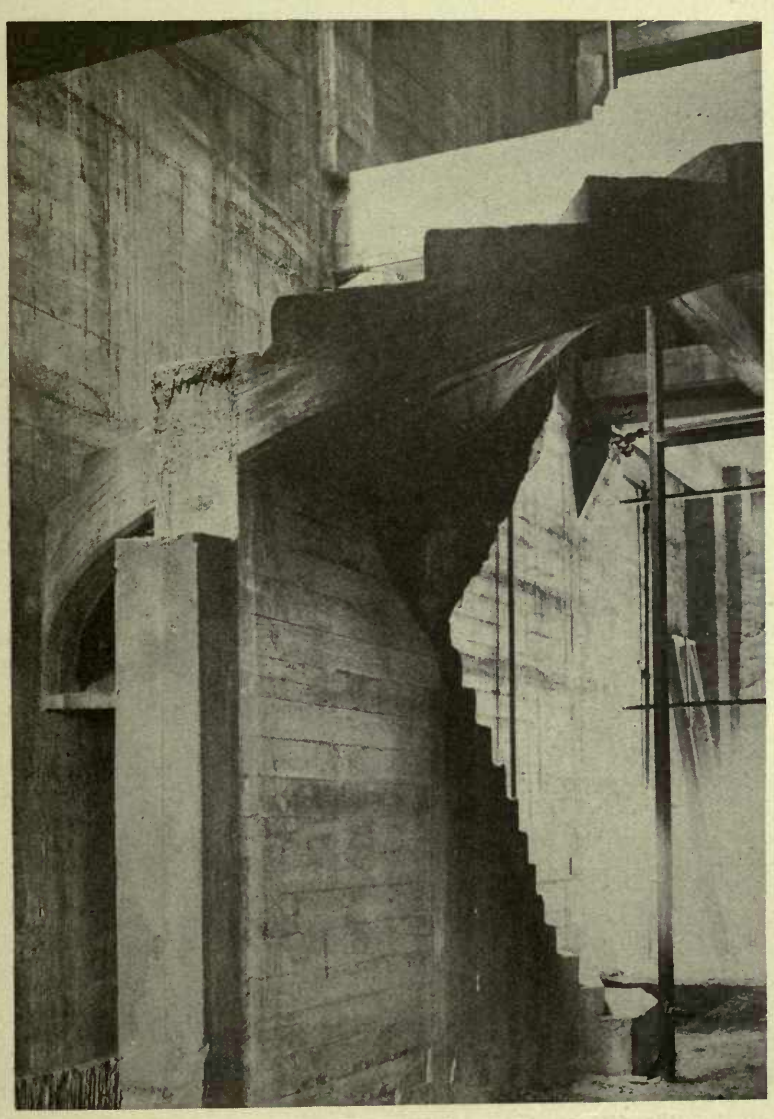


Fig. 617.—Stairway in McIntyre Building, Salt Lake City

on centres on the two side sections of each staircase, and $\frac{5}{16}$-in. rods 10 in. on centres for the middle portion.

CHIMNEY-SHAFTS

A chimney at the Northfleet works of the Associated Portland Cement Manufacturers is shown photographically by Fig. 619. It is 247 ft. high above the bottom of the foundations, and its internal diameter is 8 ft. 6 in. The foundations, 18 ft. by 18 ft., are 2 ft. thick, tapering to 4 ft. thick at the bottom, which is 35 ft. below ground level; they are reinforced by $1\frac{1}{4}$-in. by $1\frac{1}{4}$-in. by $\frac{3}{16}$-in. T's. Up to a height of 62 ft., the outer shell is 12 in. thick; there is a cavity of 4 in. and an inner shell, 4 in. thick, and above that height the chimney is a single shell, 9 in. thick. The vertical work is reinforced by steel rings $\frac{5}{8}$ in. diameter, spaced 18 in. apart, and by vertical T-bars of the same dimensions as those in the foundation. Fig. 306 (p. 197)

Fig. 618.—Staircases in Ritz-Carlton Hotel, New York

shows the template employed to keep the vertical bars in position.

In a 144-ft. 9-in. stack, reinforced with indented bars and built on a site near Drury Lane, London, by Trollope and Colls (engineer, Mr. Oscar Faber, B.Sc.), the inside diameter is only 2 ft. 6 in., and the lower portion, up to the flue entrance, is 1 ft. 3 in. thick, and the diameter at the base is 5 ft.; with brickwork, according to Mr. Faber's calculations, the least permissible diameter at the base would have been 12 ft. Above the flue entrance, the thickness of the outer shell is only 6 in. next coming an air-space 4 in. thick, and then a firebrick lining, 5 in. thick, consisting of special firebricks with a raised rib, 2 in. by $\frac{3}{4}$ in., fitting into a recess in the next course. By means of vent-holes left near the flue entrance, an air current is induced between the lining and the shell. Timber rings are embedded at intervals on the inside face of the concrete shell for the purpose of reducing to zero the stresses due to expansion, this system being in accordance with a method patented by the designer. In the firebrick lining there are special bricks at 10-ft. intervals, with side projections, between which projections and the outer shell there is a clearance of $\frac{3}{8}$ in., to allow for expansion. The foundation was taken 15 ft. below basement level, a total of 29 ft. 9 in. below ground level, and a solid block of concrete, 16 ft. 6 in. by 14 ft., was made, this serving as a foundation block and also as underpinning to existing walls. A staircase formed out of the block gives access to an engine room below the yard. Fig. 620 presents a number of sections through the shaft.

PIPES, SEWERS, AND CONDUITS

The construction of heavy pipes, sewers, etc., on the Bonna system was the subject of a report presented by Mr. Arthur E. Collins, M.Inst.C.E., Engineer to the City of Norwich, to the Sewerage Committee of that Corporation, following on a special visit paid by him to France. His report is summarised as follows: "I visited Maisons-Alfort in the Department of Seine, where a water main was being laid. The pipes were being constructed in a yard adjoining the road where they were to be laid. They had an internal diameter of 2 ft. 7 in., with thickness of walls of $2\frac{3}{8}$ in. and length of 10 ft. The length of main was five miles. The internal pressure was 120 ft. head. At about mid-thickness of the pipe walls are complete cylinders of sheet steel; these cylinders are made at the spot, and are jointed by means of a special fusing apparatus, which fuses the two surfaces of steel into one at the junction. The apparatus in question consists of a mechanically-

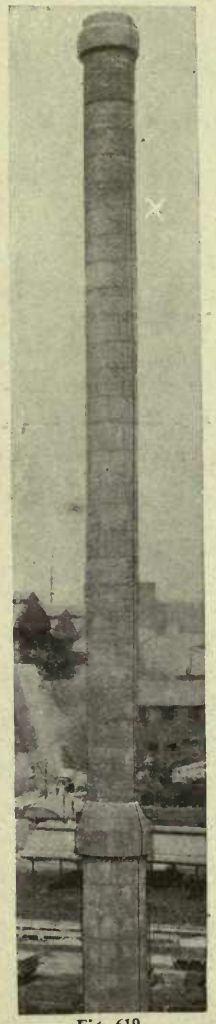


Fig. 619

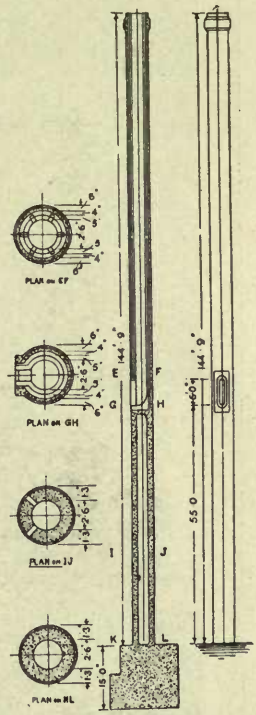


Fig. 620

Fig. 619.—247-ft. Chimney at Northfleet

Fig. 620.—144-ft. 9-in. Chimney near Drury Lane, London

operated blow-pipe, using a mixture of acetylene and oxygen. Before being fused together the edges of the sheets are turned up about $\frac{3}{8}$ in., so as to give a convenient surface upon which the fusing apparatus may operate. This arrangement also gives a flexible or expansive joint, the value of which is apparent when the next process of manufacture is seen. This consists of

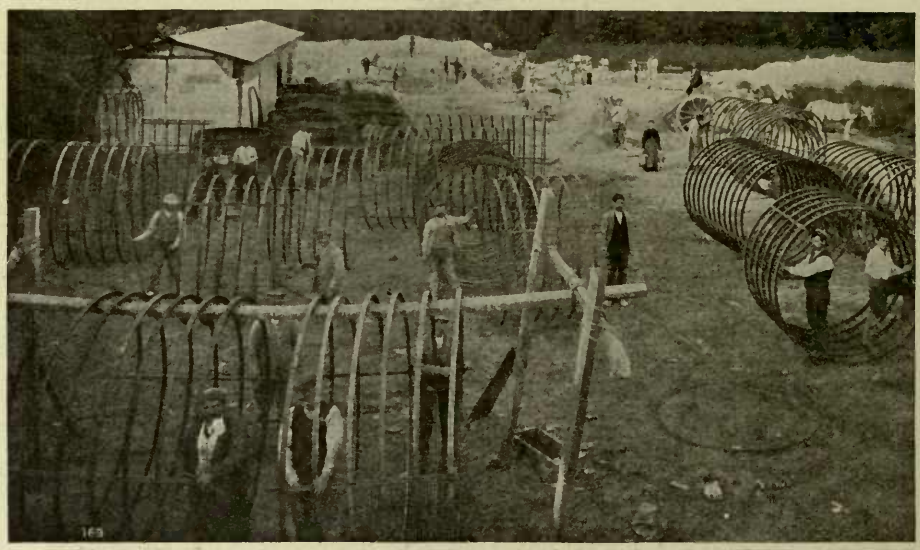


Fig. 621.—Sewer Pipe Reinforcements, Paris

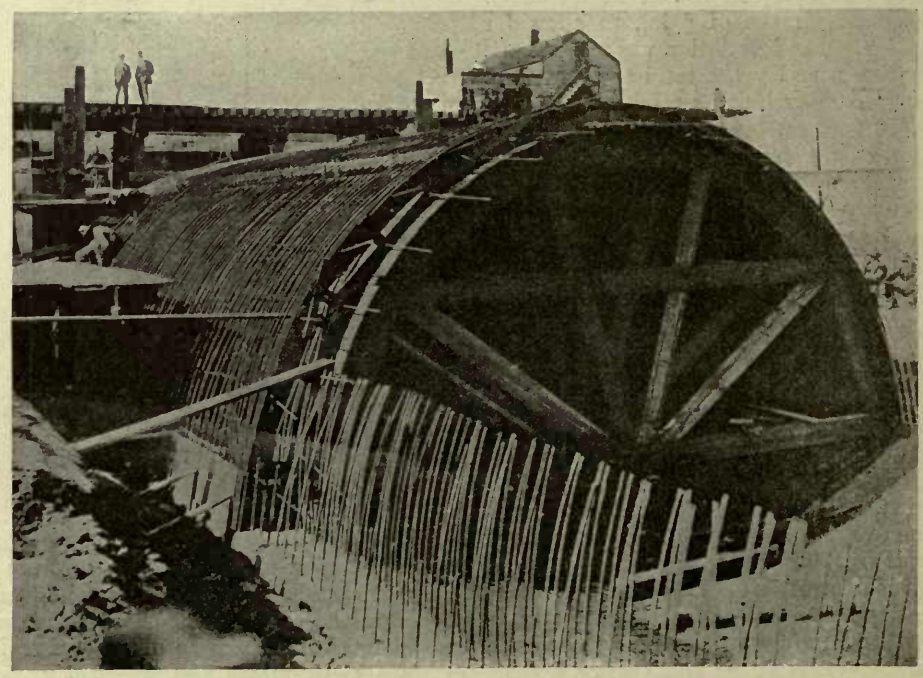
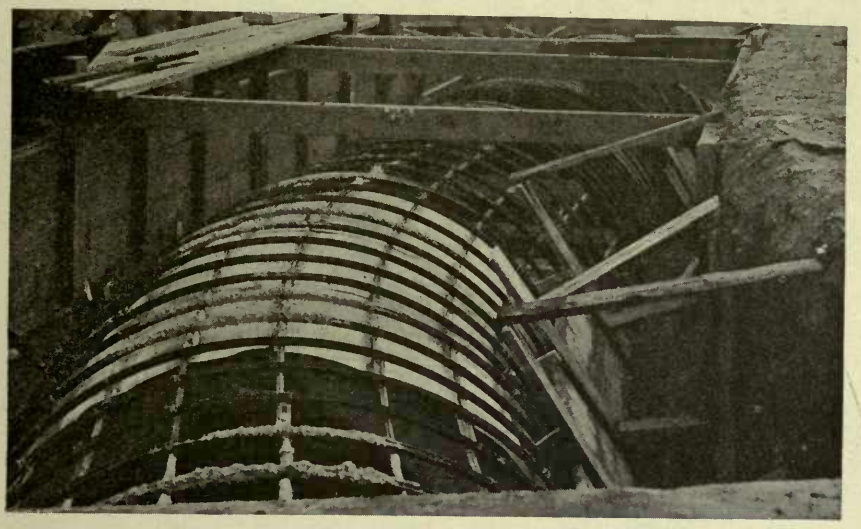
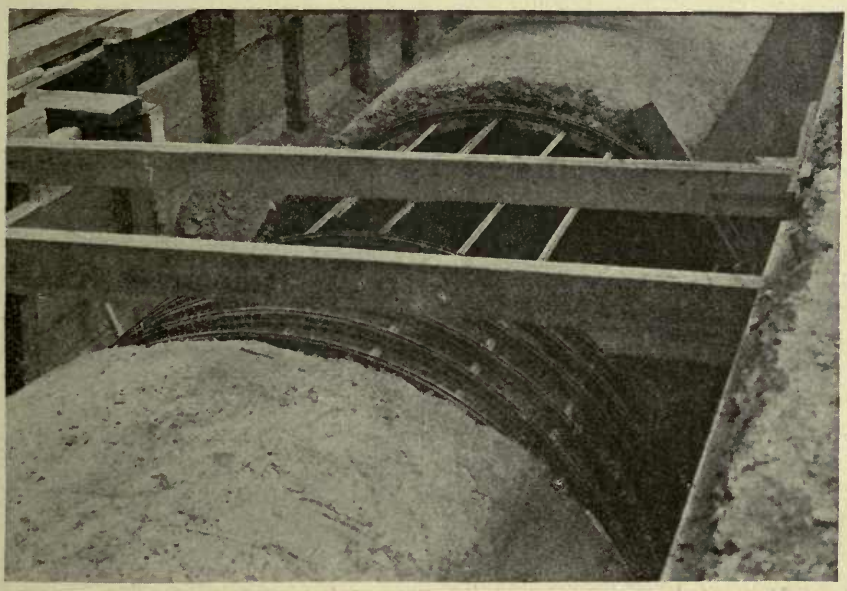


Fig. 622.—Sewer Construction at St. Louis, U.S.A.

placing within the steel tube a cage composed of steel of + section wound into a spiral with longitudinal rods at about 9-in. intervals; the longitudinal rods and the spiral are wired together at a sufficient number of intersections. The springiness of the steel tube enables it to be drawn over the internal cage, whilst at the same time it is in close adherence thereto. A similar cage, but of heavier section of steel, is drawn over the outside of the steel tube. The weight of the steel + section and the pitch of spiral depend on the diameter of the tube and the intended working pressure. The steel tube is of the same length as the pipe, and is slightly flanged at each end; the internal cage is of almost the same length as the pipe; the external cage is slightly shorter than the pipe. When the tube, with its internal and external cages, has been assembled in the manner stated, the assemblage is placed upright on a platen or table; a collapsible core of very ingenious construction is placed in position inside the





Figs. 623 and 624.—Sewer Construction at Acton, London

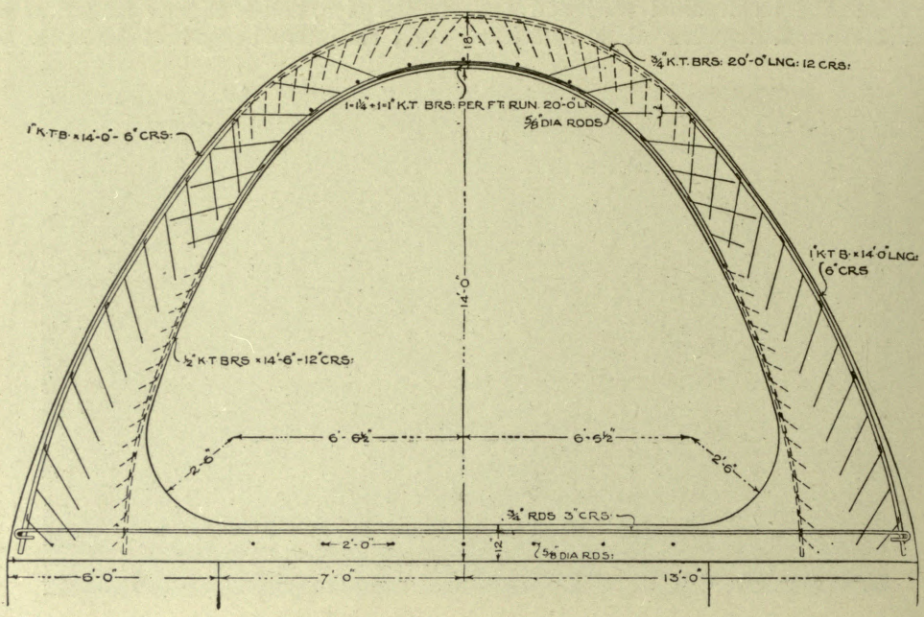


Fig. 625.—Cross Section of Culvert at Kilton

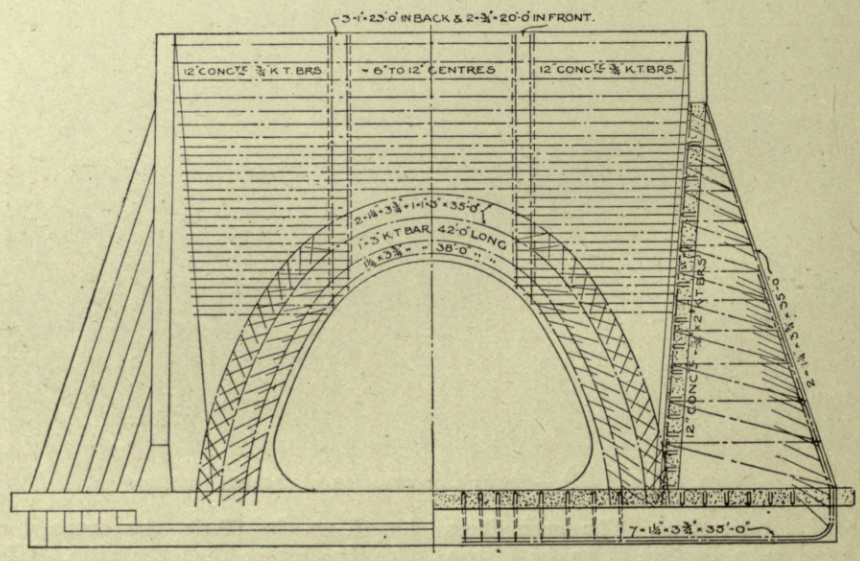


Fig. 626.—Part End Elevation and Part Cross Section of Culvert at Kilton

assemblage, leaving sufficient space all round to be occupied by concrete ; a cope in three parts is placed around the outside of the outer cage, leaving sufficient space for concrete there ; concrete is then mixed by hand

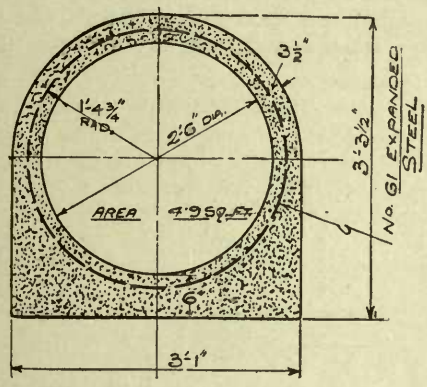


Fig. 627.—Cross Section of Conduit at Woolwich

on a concrete mixing trough on an elevated platform running on rails, and provided with cranes and other conveniences. The trough is mounted to tip to either end, and is 12 ft. by 4 ft. by 1 ft. 8 in. deep. A special mixing paddle is used somewhat similar to a small spade, but with slots in it. Four men mix the concrete dry, and six men mix it again wet. When thoroughly mixed so as to be in a practically liquid condition, by tilting the trough and opening a hatch in the lower end the concrete flows between the core and the interior of the assemblage, and between the cope and the exterior of the assemblage. During the pouring the cope is struck a series of blows with wooden mallets to assist in consolidation and in disentanglement of air."

Fig. 621 shows the reinforcements for a concrete sewer which comprises the main outfall in Paris (engineers, Bechmann and Lanney ; contractors, Messrs. Chassin and Sons). The skeleton reinforcements are clearly illustrated. On this system a pipe line, 2 kilometres (roughly 1¼ miles) long was constructed, the diameter being 2 m. (nearly 79 in.).

Fig. 622 shows a sewer at St. Louis, U.S.A., in course of construction. The immensity of the work can be gauged from the heavy centering employed. The reinforcement was by means of indented bars.

Figs. 623 and 624 show a sewer constructed for the Urban District Council of Acton by the Columbian Fireproofing Company. It is a pipe 4 ft. 6 in. in diameter, constructed on the " Bonna " system. The pipe has a semi-circular crown with a segmental invert, this shape having been adopted because the sewer lies near the surface of the road, and it was desired to give the pipe as much strength as possible without diminishing the bore. The thickness of the concrete at the crown is 6 in., the reinforcement being 2-in. cruciform bars.

Fig. 346 (p. 210) is a photographic view of a conduit at Woolwich in course of erection, the illustration showing the Blaw collapsible steel centering being pulled forward for the next section. Fig. 267 shows a cross section of the pipe. For concreting the invert, the half-round centres, 2 ft. 6 in. in diameter, were set up on a dish of concrete, and the work was then concreted up to the springing line. Then for concreting the crown, as soon as the work would allow, the centres were reversed and set up on timbers, about 20 ft. of pipe being done at a time, the centres being then collapsed and pulled forward on ponding angle iron is placed on the centre, angle irons as illustrated on p. 210. A corres- and the rolling forward is made possible by means of balls 1⅛ in. in diameter.

Drawings of a reinforced concrete culvert at Kilton are presented by Figs. 625 and 626 ; this culvert was built under the North Eastern railway line between Skinningrove and Kilton, to provide a waterway at the

Fig. 628.—Wharf at Dennemont

bottom of a valley formerly spanned by a viaduct. The work was designed by the Trussed Concrete Steel Company, Ltd., under the supervision of Mr. W. J. Cudworth, late chief engineer of the North Eastern Railway. The reinforced concrete at the crown of the culvert is 18 in. thick, and the total length of the culvert is 450 ft.

WHARF AT DENNEMONT

A wharf at Dennemont, near Mantes, built on the banks of the Seine, at the works of a cement manufacturer, is shown in Figs. 628 and 629, the construction being according to the Coignet system, and the designer being E. Candlot. The wharf is 8·75 m. long and 9·5 m. wide from front to back, the decking being 3·7 m. above normal water level. It is carried on twelve reinforced concrete piles, each 30 cm. in diameter, placed in three rows, twelve columns being built up from the piles. At a uniform level at the tops of the piles there are both longitudinal and transverse members, and there are smaller members at the tops of the

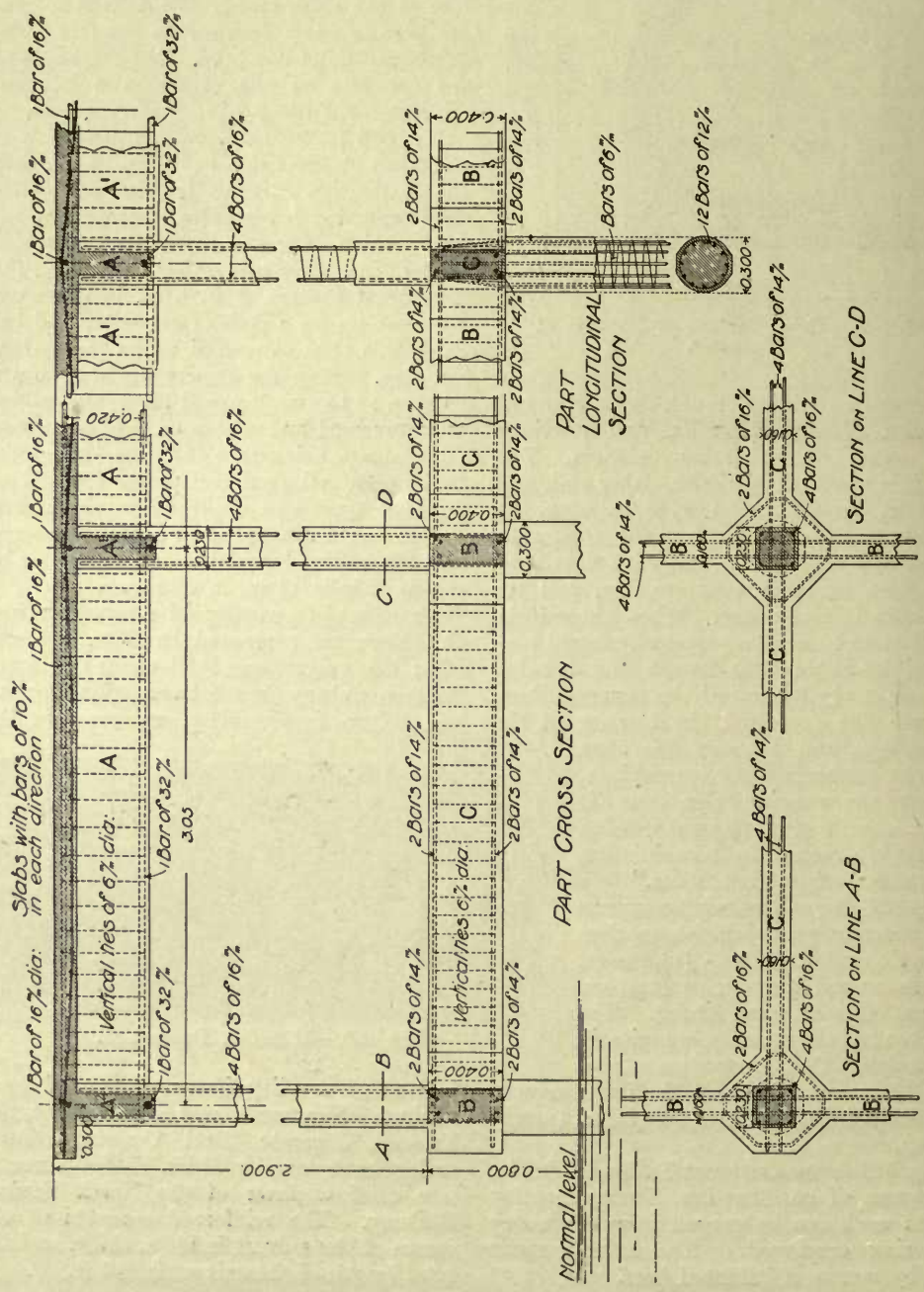


Fig. 629.—Details of Wharf at Dennemont

columns, the latter members supporting the decking. The horizontal members between the tops of the piles are 16 cm. wide and 40 cm. deep, and are reinforced by four bars of 14 mm. diameter, two near the top and two near the bottom surface of the concrete. These members are not designed to act as beams, but as struts, and it will be noted that there are vertical reinforcements along these struts consisting of steel wire 6 mm. in diameter and spaced 10 cm. apart. The construction of the piles, of the columns over the piles, of the lower horizontal members, and of the top beams and decking is well shown in Fig. 629. The columns, 23 cm. square, contain four vertical reinforcements, each 16 mm. in diameter, these being wound spirally with 6-mm. wire, the coils being 10 cm. apart centre to centre, as indicated in the sectional views. The deck beams have a lower reinforcement of a 32-mm. diameter bar and an upper reinforcement of a 16-mm. bar, the two being connected by vertical ties of 6-mm. wire. The decking panels are reinforced by sixteen 10-mm. bars, half of them at right angles to the others, curved upwards to pass over the top reinforcement of the main beams.

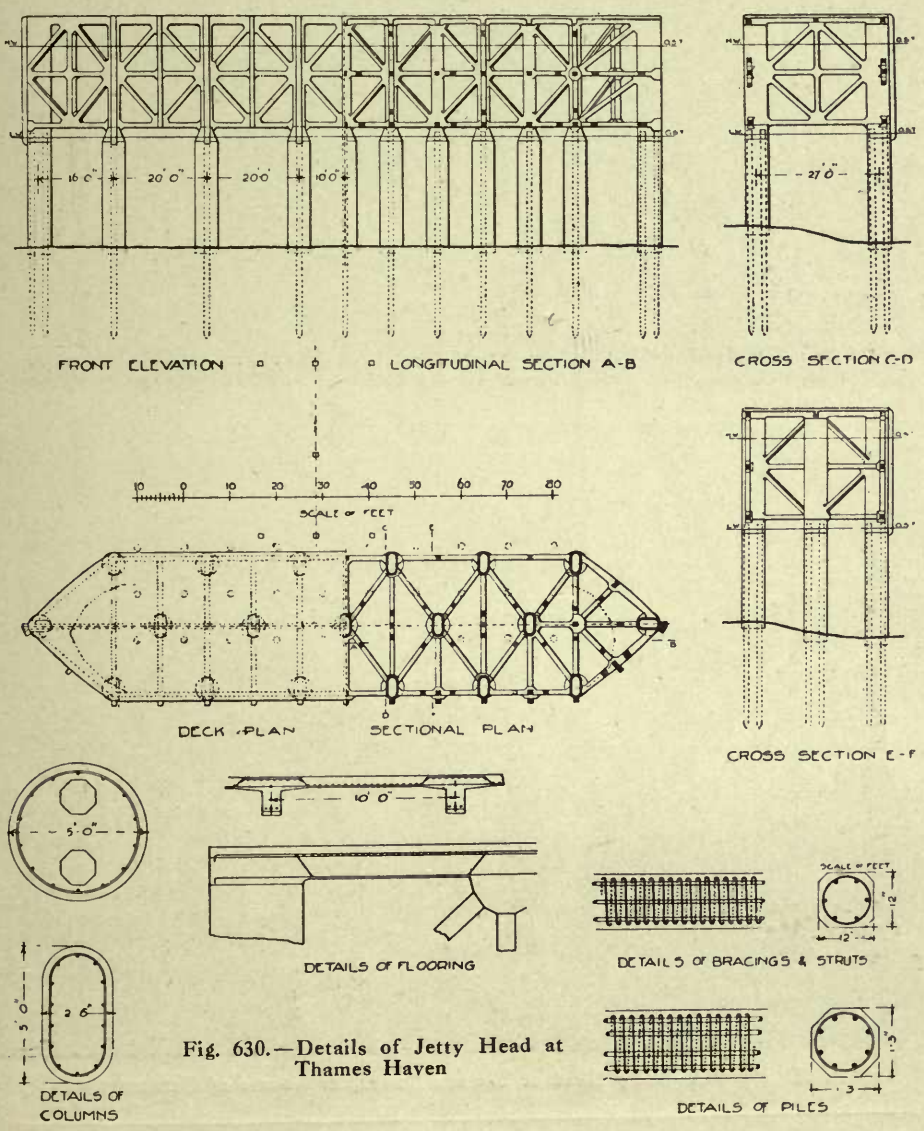
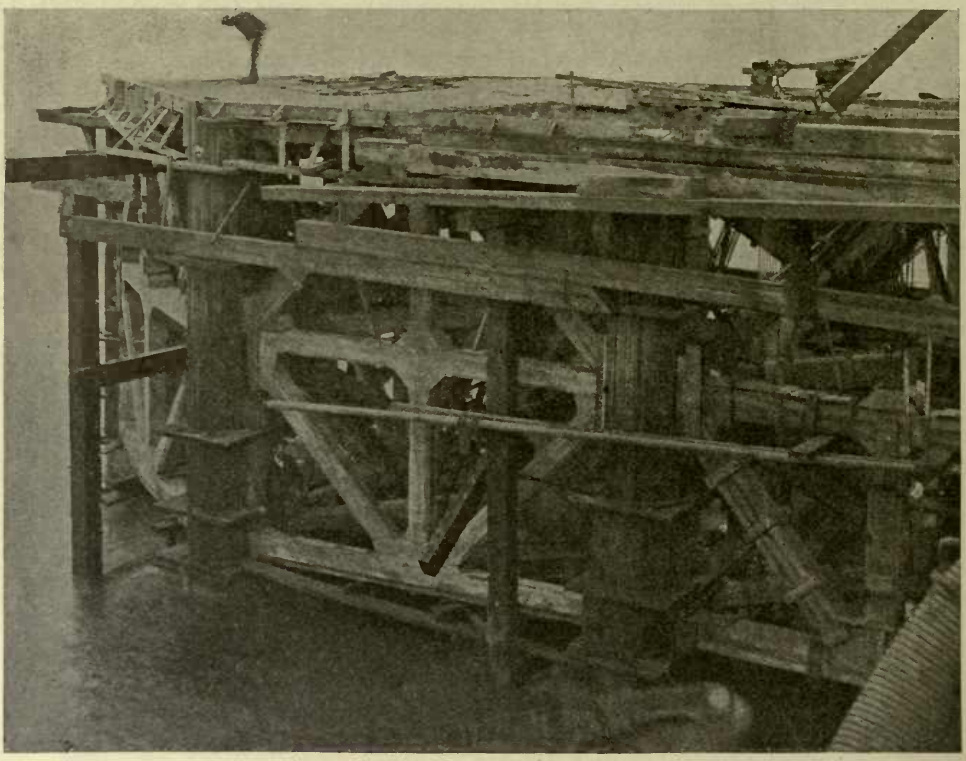
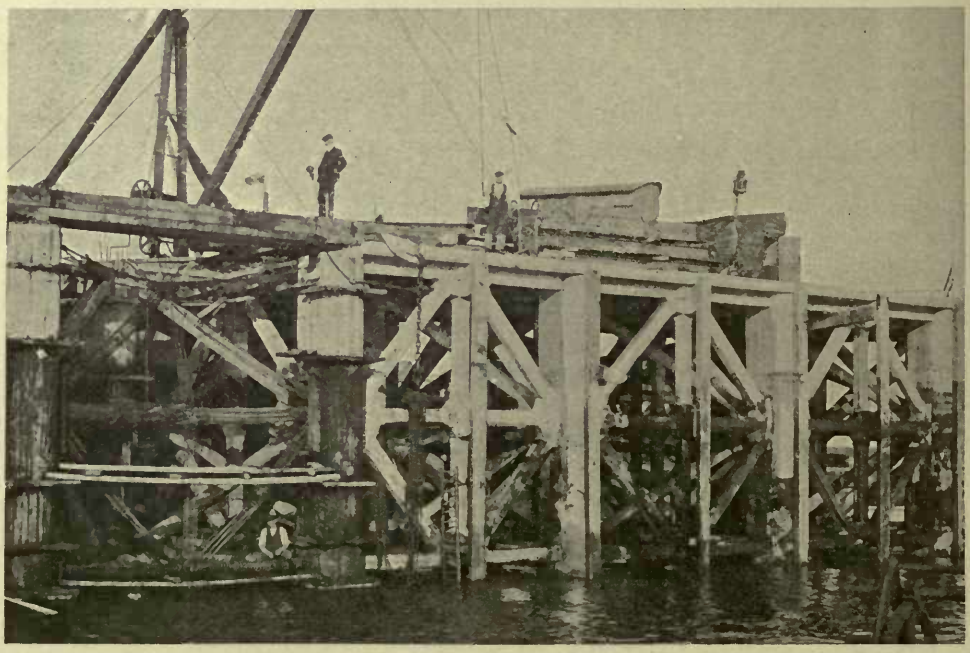


Fig. 630.—Details of Jetty Head at Thames Haven

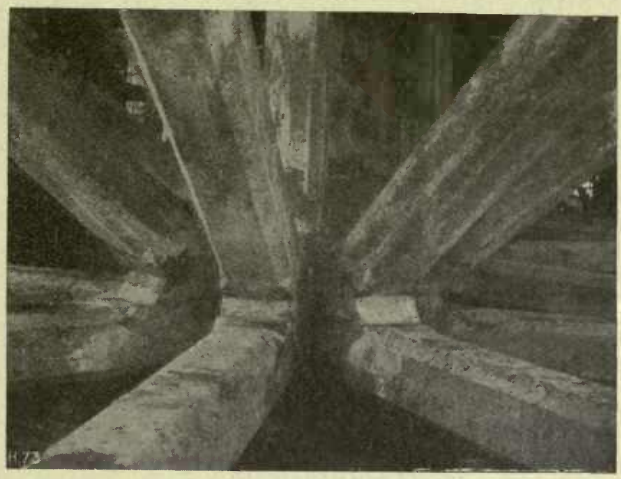


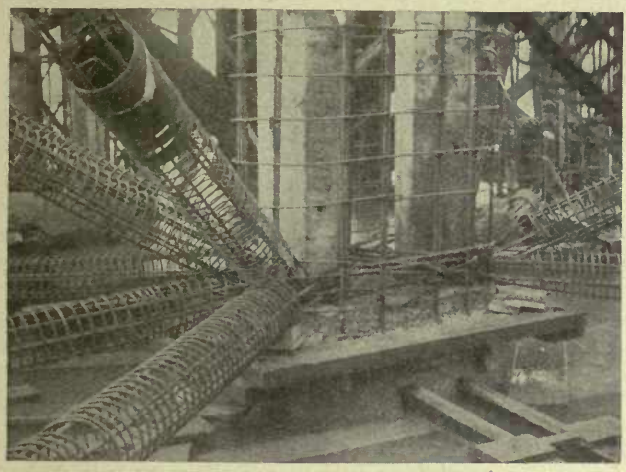


Figs. 631 and 632.—Jetty Head at Thames Haven, in Course of Construction

JETTY HEAD AT THAMES HAVEN

Detail illustrations of a jetty head at Thames Haven are given by Figs. 630 to 634. This jetty replaces an old T-head composed of cast-iron piles with wrought-iron bracings, a great part of which was carried away as the result of a collision with a large vessel, what was left of the structure, with the addition of temporary wooden piles, serving as a stage from which to construct a new jetty head in reinforced concrete. The work in course of progress is shown by Fig. 631, in which illustration some of the centering still in position will be noticed. More of the centering round the piles and bracing is illustrated in Fig. 632, whilst piles and bracing are shown by Fig. 633, the skeleton reinforcement for this work being shown by Fig. 634. The following description of the work is quoted from *Concrete:* "The jetty head is 136 ft. long by 32 ft. wide. The deck level is 5 ft. 6 in. above high-water spring tides, and about 45 ft. to 48 ft. from the bed of the river; the spring tide range is about 18 ft. Vessels up to 7,000 tons or 8,000 tons burden are accommodated, so that the structure was needed to be specially strong in cross section, in order to take the shock resulting from such large vessels. The jetty head was required to be designed as a complete structure in itself and independent of the approach jetty, which could not be reckoned on for support. It being impossible to fix concrete bracing below low-water level, it was decided to make the main support of the structure composed of two piles, which were surrounded by a reinforced concrete cylinder 5 ft. in diameter, and then filled in solid, so as to give great lateral stiffness to each support. A general plan and elevation is attached herewith. There are three main columns in the width of the structure, the centre column being midway between the outer columns, with horizontal and diagonal bracing in all directions. It will be seen that this method of construction and bracing has the effect of distributing all shocks over a large number of the columns, and thereby giving great rigidity to the structure.





Figs. 633 and 634.—Piles and Bracing of Thames Haven Jetty Head

"The columns are 5 ft. in diameter up to low-water line, after which they are reduced to an oval section of about 5 ft. by 2 ft. 6 in., and continued in this section up to the under-side of the deck. All the horizontals and diagonals are also spirally and longitudinally reinforced, so as to take stresses of either compression or tension. Timber fenders are bolted to the columns to act as rubbers against the jars of ships.

"The method of procedure in carrying out the work was as follows: Piles 45 ft. long

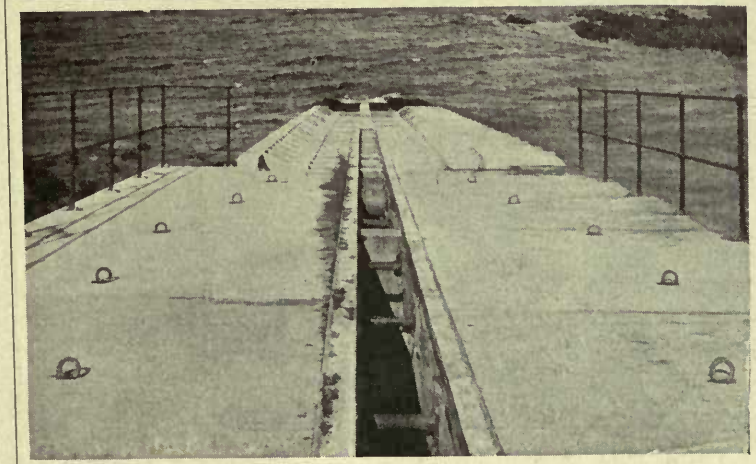
were driven 18 ft. to 20 ft. into the bed of the river in groups of two, as shown on the plan. The temporary steel casing was then dropped over the piles and bedded about 2 ft. or 3 ft. into the bed of the river. Inside this steel casing and over the piles an enveloping reinforcing, composed of longitudinal rods and hoops, was dropped, and the casing was filled up with concrete 3 to 1, deposited through the water by means of shoots, to 3 ft. above low-water level. The columns, braces, and deck were then built up in situ with the usual timber shuttering.

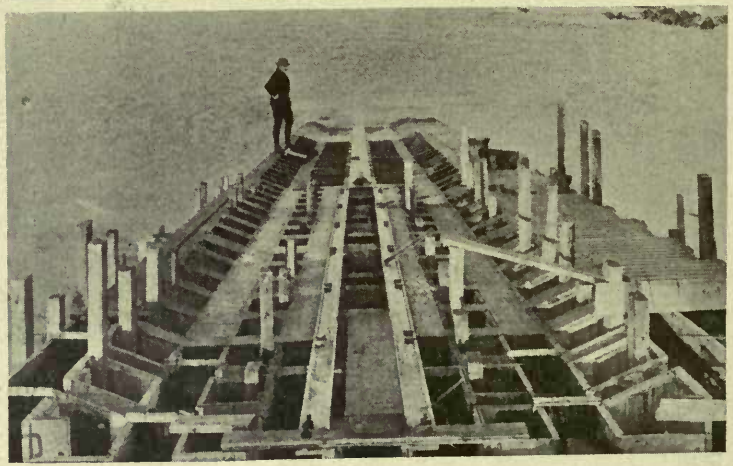
"The temporary steel casing was made in two halves, with vertical joints bolted together, so that it could afterwards be taken away, when the concrete was set, by means of a diver unbolting the vertical joints. The piles were 15-in. diameter octagonal section, made on the Considère principle; the reinforcing was composed of ⅜-in. diameter steel, spirally wound with 2-in. pitch, and eight longitudinal rods of 1-in. diameter. The ground into which the piles were driven is composed of Thames ballast with an overlay of about 4 ft. to 6 ft. of soft, sandy mud. The piles were driven with a 30-cwt. monkey, falling 6 ft. to 7 ft., mostly with a timber dolly intervening between the head of the pile and the monkey, but in some cases with the monkey falling direct on to the head of the pile. The piles withstood this driving very well indeed, and, where the dolly was used, the head did not suffer in the least, although, in some cases, the piles were driven to a set of 100 blows for the last inch before they were regarded as being driven to refusal. In the cases of direct driving, without any dolly at all, the piles were simply broomed up for about 12 in. of the head in a similar manner to timber piles."

LIFEBOAT SLIPWAY

The photographs (Figs. 635 to 639) show a reinforced concrete slipway that has been built at the Royal National Lifeboat Institution's station at Ackergill, near Wick, North Scotland. The illustrations show the various stages in the construction, and give much information as to the manner in which the concrete is reinforced; in particular, the spiral reinforcement of the columns and bracings will be noted. The total length of the slipway is 194 ft., and, excepting for the mass concrete toe in the sea, the whole of the construction is of reinforced concrete. The octagonal columns, 20 ft. apart, are 14 in. in diameter, and have bases 2 ft. 6 in. square, which go down 2 ft. into the solid rock. The extra precaution was taken of boring holes in the rock, so that the reinforcing rods go down for a further 18 in. and

Fig. 635.—Lifeboat Slipway, Ackergill





Figs. 636 to 639.—Lifeboat Slipway, Ackergill

are cement grouted into place. The platform, which is well shown in one of the photographs on p. 379, accommodates the lifeboat and the hauling winch; it is carried 16 ft. above the shore on three rows of columns, and is 58 ft. long, 16 ft. wide, and the slabs that form the decking are 5 in. thick. The sloping part of the slipway has a gradient of 1 in 7. To facilitate the moving of the lifeboat, there are longitudinal keelway beams in pairs, these being 12 in. wide and 20 in. deep, between which are a number of rollers to support the keel of the boat.

ROOFED RESERVOIR

Figs. 640 and 641 show part of a service reservoir at Kloof Nek, Cape Town, 900 ft. above sea level. The work was carried out by the engineering staff of the Cape Town City Council, Messrs. Murray and Stewart being the contractors and the reinforcement being expanded metal. The ninety-six columns are of rectangular cross section varying from 18 in. by 18 in. to 30 in. by 18 in.; some of them are shown in the photographic view (Fig. 640), from which it will be understood that the reservoir is circular in plan. The inside diameter is 162 ft., the height to top-water level is 24 ft., and the total capacity is 3,000,000 gallons. An outline diagram, showing the rectangular columns and the arched roofing is presented by Fig. 641. The columns are reinforced by $\frac{5}{8}$-in. diameter rods at each angle, spread outwards into the concrete base, the rods being wired together with $\frac{1}{8}$-in. diameter wire, the pitch of the spiral winding being 9 in. In plan the columns form five concentric circles, 15 ft. apart, and they serve as supports for the semi-circular arched walls, which are level at the top, to carry the roof, which is constituted of segmental arches springing radially from a central dome to the concentric walls in series, the outer arch, as shown in the diagram, merging into the main outer wall. The roof is reinforced

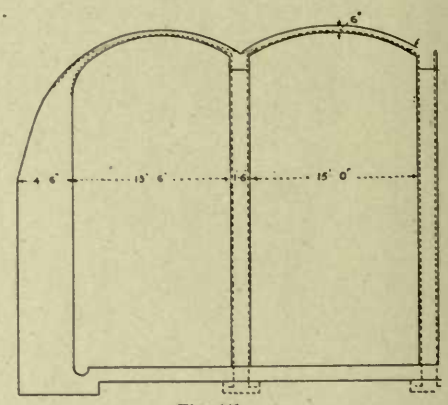


Fig. 641

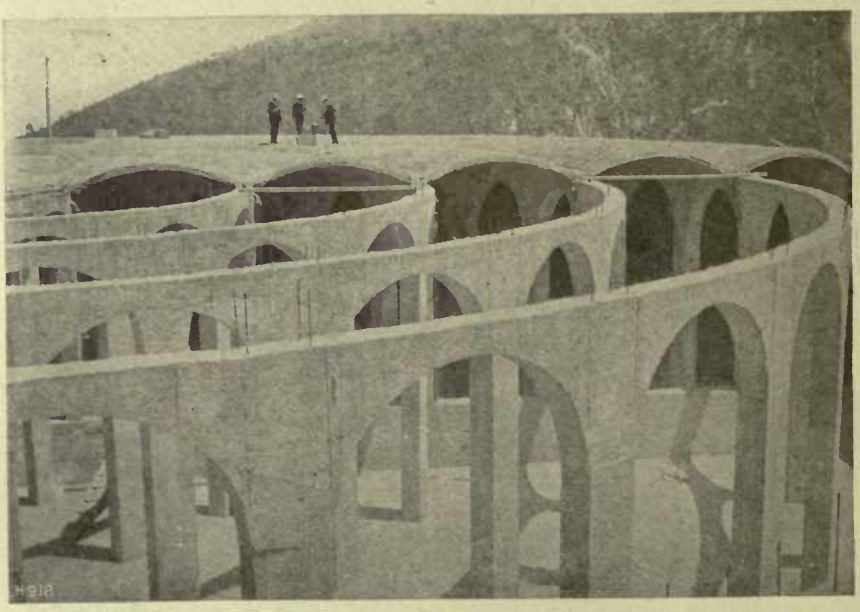


Fig. 640 and 641.—Columns and Arched Roofing of Kloof Nek Reservoir

with expanded metal of 3-in. mesh laid with a minimum lap of 6 in. The reinforcement is laid as near the intrados of the arches as possible, and there is a further layer of expanded metal near the extrados of the haunches of the segmental arches, and turned in to meet the reinforcement at the intrados. The rise of the arches is 2 ft., the minimum thickness 6 in., while the thickness at the springing is 9 in. The concrete used throughout consisted of 1 of cement, 2 of sand, and 4 of broken granite, the latter not exceeding 1 in. for the roof arches and 1½ in. for the rest of the work. Stone obtained at a quarry one-third of a mile distant from the reservoir was used. At the vertical joints the usual V-shaped bond was strengthened by inserting two 1-in. steel rods, 4 ft. long, for a distance of 2 ft. on each side of the joint. In view of the known porosity of concrete, the method of finishing is of interest. The whole of the inside of the reservoir, to a height above water level, as well as the extrados of the roof arches, was hacked, thoroughly cleaned down, and then rendered in cement mortar of 2 to 1 with a thin skimming of neat cement. The roof of the reservoir is covered with earth to a minimum thickness of 15 in.

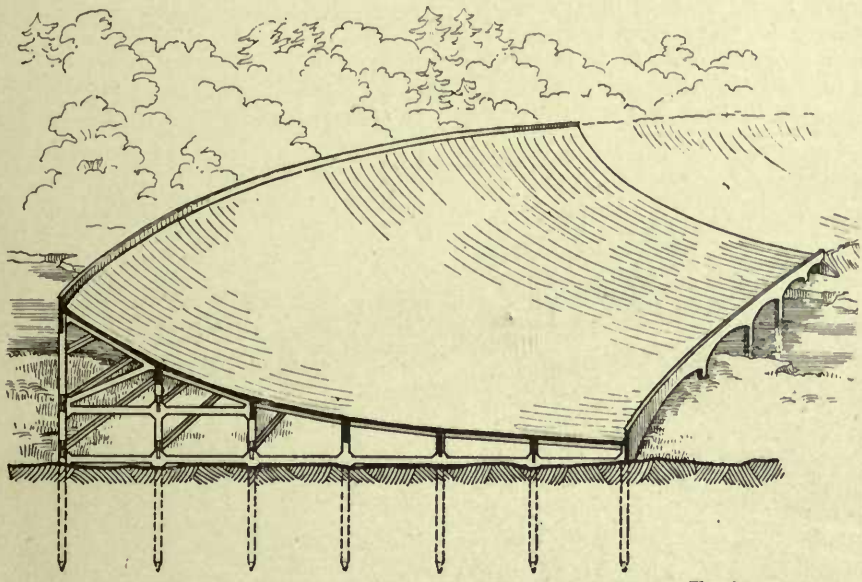


Fig. 642.—Bridge forming part of Brooklands Motor Track

SPECTATORS' STANDS, ETC.

While coming under the heading of stands, etc., the particular construction illustrated by the part perspective and part sectional view shown by Fig. 642 is actually a bridge of peculiarly distorted shape. It forms part of the Brooklands motor track at Weybridge, at the place where the track crosses the river Wey. It was necessary to make provision for a considerable volume of overflow water which comes down occasionally. Viewed in plan, the track exhibits two curves, one with a radius of 1,000 ft., and the other with a radius of 2,000 ft., and at one end of the bridge the outer edge of the track is banked up to the height of 40 ft. above the datum line, which is from 5 ft. to 8 ft. below normal ground level, whilst the inner edge is only 12 ft. above the datum line; at the other end of the bridge the corresponding heights above the datum line are 32 ft. and 10 ft. 6 in. The bridge has a length of nearly 200 ft., and provides a track which is uniformly 100 ft. wide. The bridge was designed to carry a super load of 112 lb. per square foot, and a rolling load of 2 tons per square foot. The reinforced concrete construction was on the Hennebique system, and the foundation includes forty-two piles, mainly 14 in. square, their heads being in monolithic connection with the beams and columns of the superstructure. The tops of the columns are joined by longitudinal and cross-beams, whose upper surfaces were curved so as to be adapted to the concave road decking, this

consisting of slabs from 4½ in. to 6½ in. thick, sub-divided by secondary beams into panels roughly 17 ft. by 5 ft. To hold up the earthen embankment at one end of the structure, there are panels of reinforced concrete, and the end retaining wall is extended by means of wing walls in order to prevent the material flowing down towards the channel of the river. The construction at the lower end of the bridge is much the same, but the section illustrated is taken on a line which precludes the panelling, etc., being shown.

Fig. 643 shows the Stadium at Shepherd's Bush, London, in almost a finished state, the roofing being in course of construction. It is interesting to note in connection with this structure that at an early stage it was intended to build it entirely of reinforced concrete, but that ultimately the stand was built in steel, and only the platforms were of reinforced concrete. For the purpose of placing this in position there was an inclined tramway running up the Stadium. The reinforcement consisted of indented bars, ⅓ in. square, the bars being placed 12 in. apart, and every fourth one being carried through the channels and turned over as shown in Fig. 644 in accordance with a request of the London County Council. The slabs are 2 in. thick, and the concrete for this consisted of 1 part of cement to 4 parts of an aggregate consisting of Thames ballast crushed and graded to pass a ½-in. ring, with the addition of one-third sand. The timber seats are bolted to studs embedded 1 in. in the concrete, an arrangement that has given complete satisfaction.

A view of a stand erected for the Bradford City Football Club, Ltd., at Midland Road, Bradford, is shown by Fig. 645, the photograph having been taken in the course of construction, and Figs. 646 to 650 are some of the working drawings. The stand is a striking example of open framework construction, the pillars being sufficiently strong to withstand lateral stresses, and thus rendering unnecessary the employment of diagonal struts; the horizontal struts for bracing the pillars together are "symmetric beams." The architect was Mr. Archibald Leitch, M.I.Mech.E., the working drawings of the reinforced concrete were got out by Messrs. F. A. Macdonald and Partners, Kahn steel reinforcement was used, and the responsible contractors were Messrs. John Ellis and Sons, Ltd., of Leicester. With the exception of the roof the whole of the construction is of reinforced

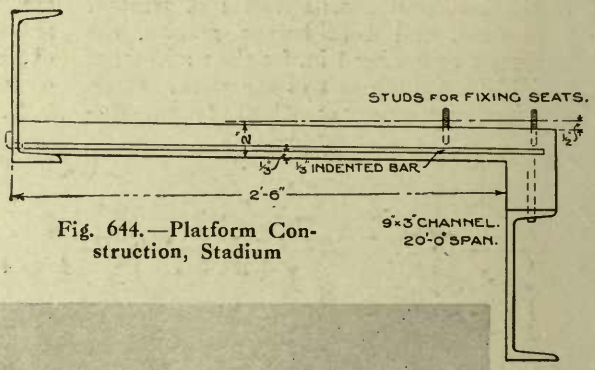


Fig. 644.—Platform Construction, Stadium

Fig. 643.—Stadium at Shepherd's Bush, London

Fig. 645.—Bradford Football Stand, in Course of Construction

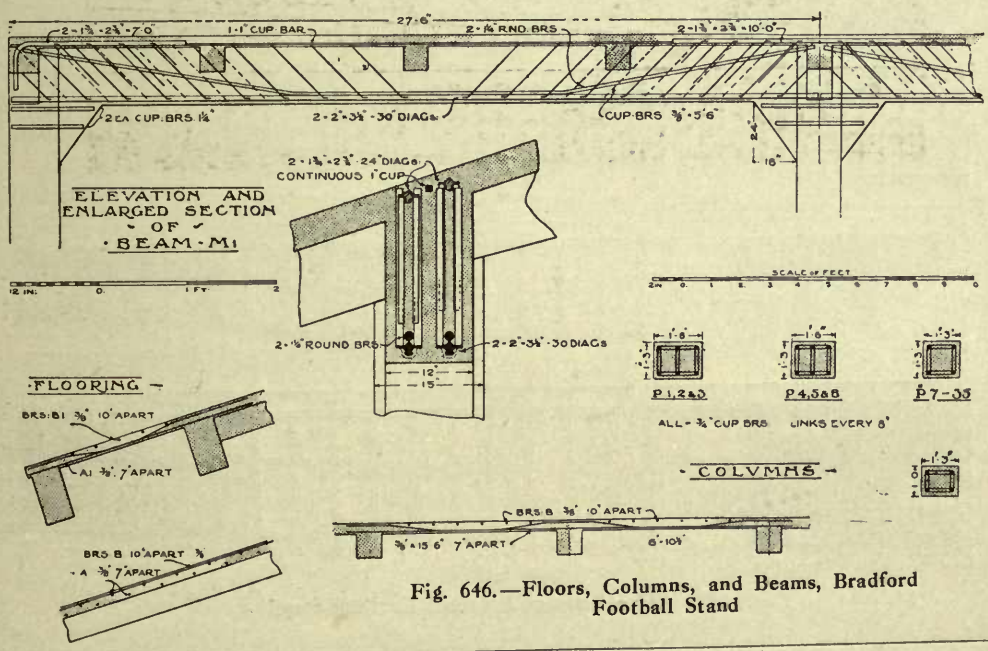


Fig. 646.—Floors, Columns, and Beams, Bradford Football Stand

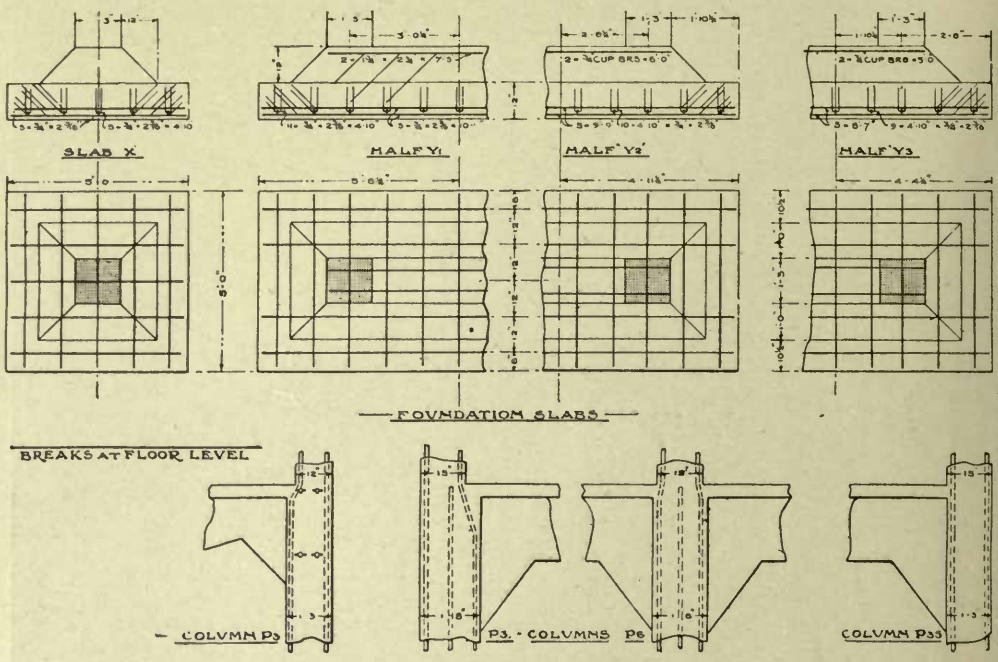


Fig. 647.—Details of Foundations and Columns, Bradford Football Stand

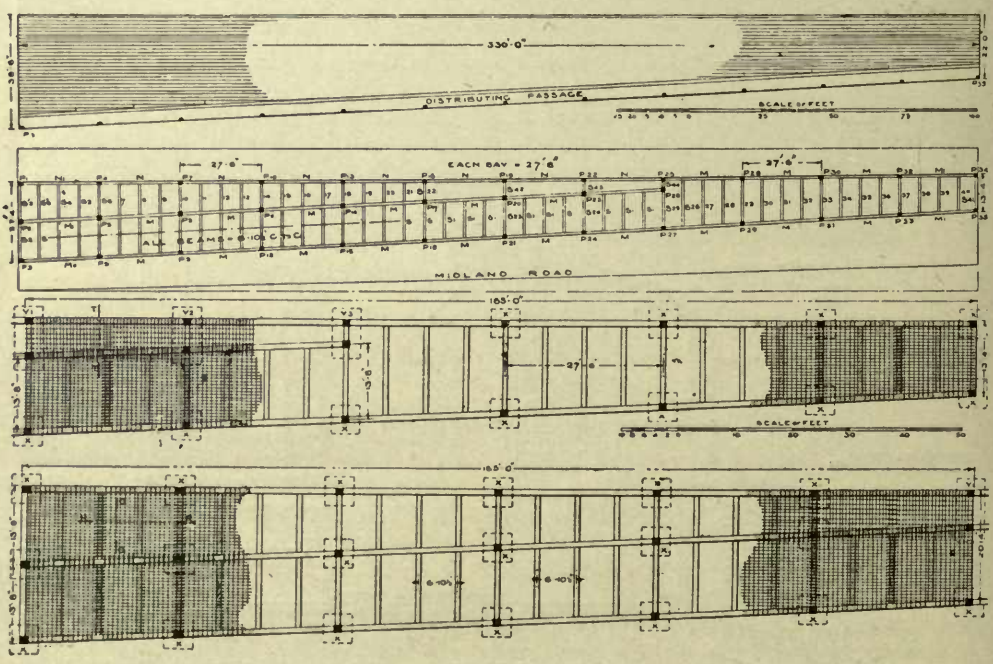


Fig. 648.—Plans of Bradford Football Stand

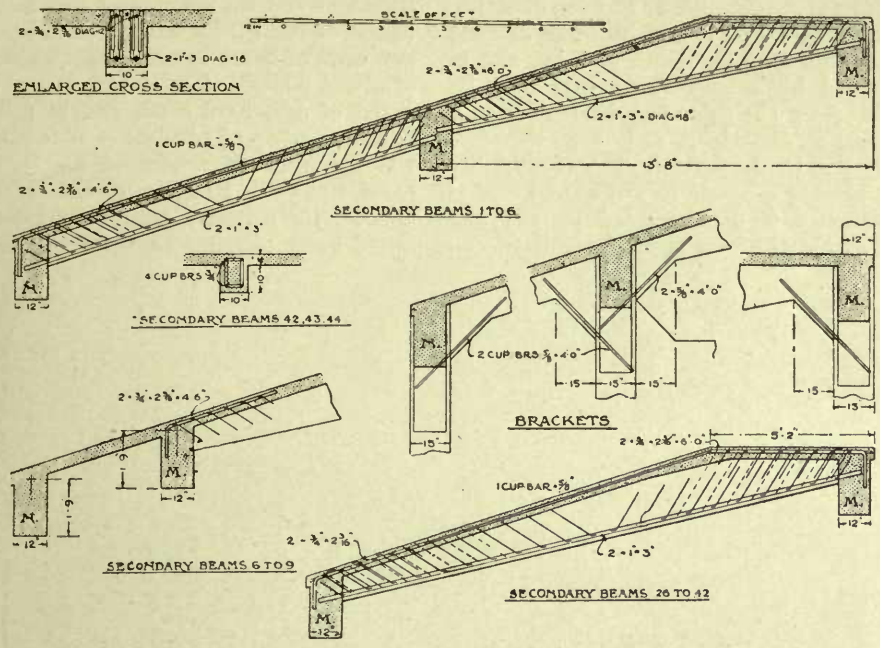


Fig. 649.—Details of Beams and Brackets, Bradford Football Stand

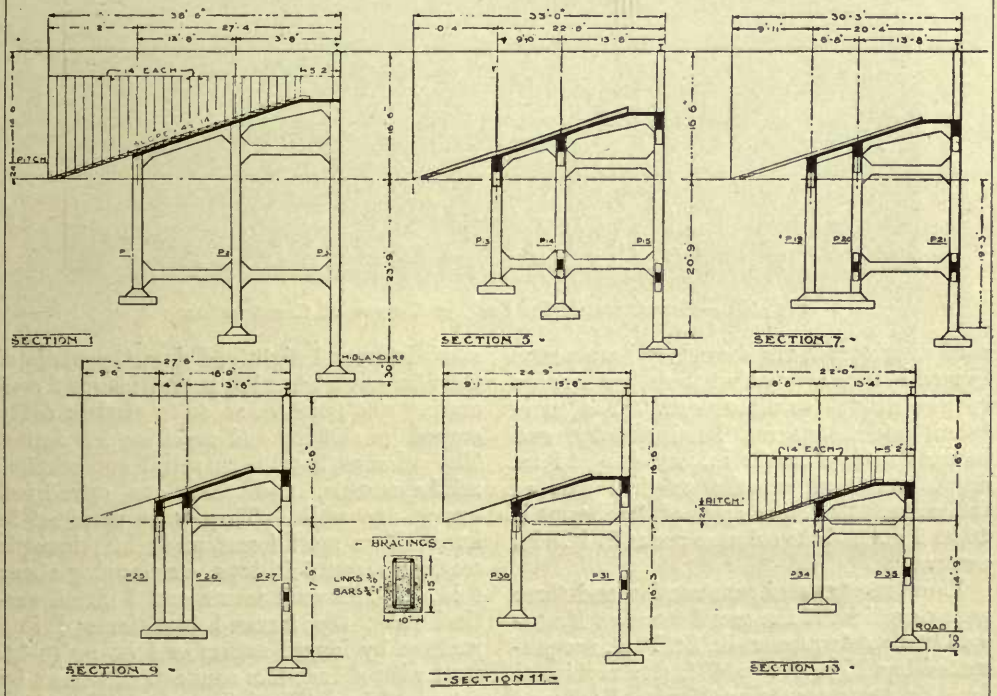


Fig. 650.—Various Sections through Bradford Football Stand

concrete, the length of the stand being 330 ft., and the breadth 27 ft. 4 in., diminishing uniformly to 13 ft. 4 in. The decking slab has an effective depth of 4 in., and is reinforced by a network of bars over its whole area; the secondary beams are in a transverse direction, and are therefore inclined to the horizontal. The adaptability of reinforced concrete to awkward forms of construction is demonstrated by the fact that as the platform is partly inclined and partly horizontal, the secondary beams were formed in such a way that their depth varies in the length, as illustrated. The main beams and lintels run longitudinally, and in section they are 12 in. broad by 21 in. deep, measured on the vertical axis of the beam. The clear span of the beams is 26 ft. 3 in., and the columns are at 27 ft. 6 in. centres.

Views in course of construction and three part elevations of the grand stand at Minnesota State Fair race track, St. Paul, are presented by Figs. 651 to 654, the architects for this structure being Messrs. Reed and Stem, of St. Paul, the reinforcement being by means of Kahn bars. The stand has a length of 800 ft., a width of 240 ft., and a height of 105 ft., its sixty tiers of seats and two rows of boxes having a seating capacity of about 30,000. The soil on the site consisted of dry, hard sand, and in this were dug a number of pits about 8 ft. deep, in which were built reinforced concrete piers —some of them 10 ft. wide—to act as footings for the columns, which numbered 156. Speed being of prime importance, work was carried forward night and day, the whole of the false-work being left in position until concreting was finished, so as to obviate delay caused by shifting and repairing the forms. The columns have both spiral and vertical reinforcements, and they are cylindrical except the walls. They were moulded in knock-down steel forms made in telescopic sections about 5 ft. long, overlapping about 4 in. on horizontal seams and 3 in. on vertical ones, the forms being clamped into position by hinged collars of 4 in. by $\frac{3}{16}$-in. steel with a hasp-like connection, locked by a tapered key. The joints were stiffened by wooden fillers 3 in. thick, inserted between

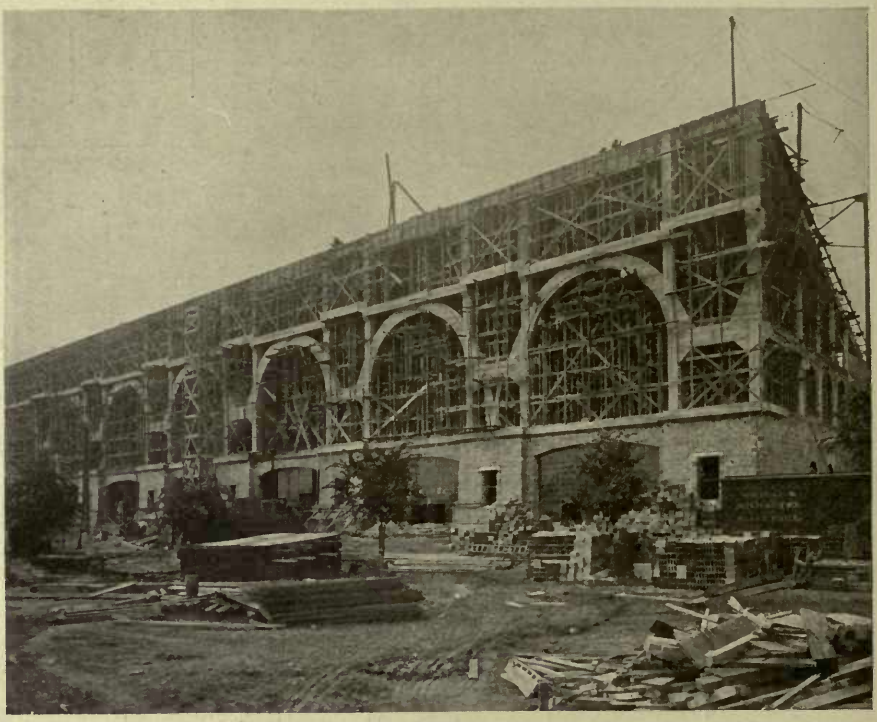


Fig. 651.—Grand Stand, St. Paul, in Course of Construction

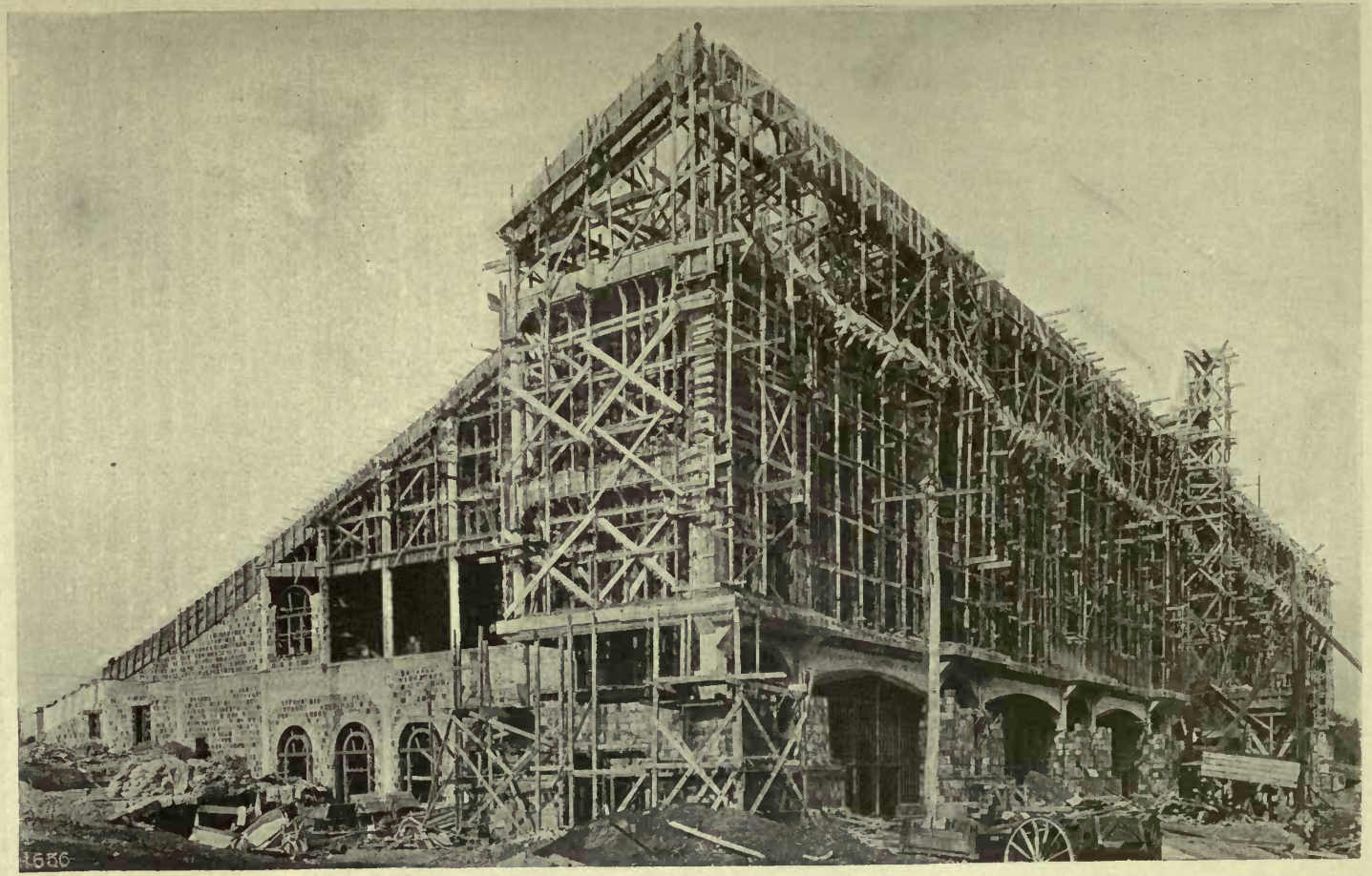


Fig. 652.—Another View of Grand Stand, St. Paul, in Course of Construction

the collars and the steel sheets. The knee-brace sections between the columns and beams were made by extending the beam moulds. The projecting reinforcements from the foundation piers were connected to the vertical reinforcements of the columns by means of sleeves 2 ft. long, the same method of joining being employed at the various floor levels between the columns of one story and those of the next. The method of carrying on the work was in every sense expeditious and up-to-date, and the extent to which the concrete was placed in position without hand labour was remarkable. Concrete was hoisted in buckets, automatically dumped into cars running on a 24-in. gauge Koppel service track, and conveyed to the work and shot into the forms through tapered shoots, the only hand work being spreading and light tamping. Terrace steps in the upper platform were cast on corrugated iron sheets bent at right angles to take the tread and riser and supported at frequent intervals on horizontal cross pieces forming parts of triangular offsets nailed to the upper sides of false-work beams inclined parallel to the slope of the platform. At first the weight of the wet concrete produced a heavy downward lateral thrust, which deflected the riser forms; but this was corrected by the use of a tier of very heavy upper inclined beams, to which the riser forms were secured in the same way as those of the supporting forms below. The lower ends of the upper inclined beams engaged a heavy horizontal girder, tied back at frequent intervals by horizontal tension rods, which transferred the thrust across the building to heavy columns or other suitable members.

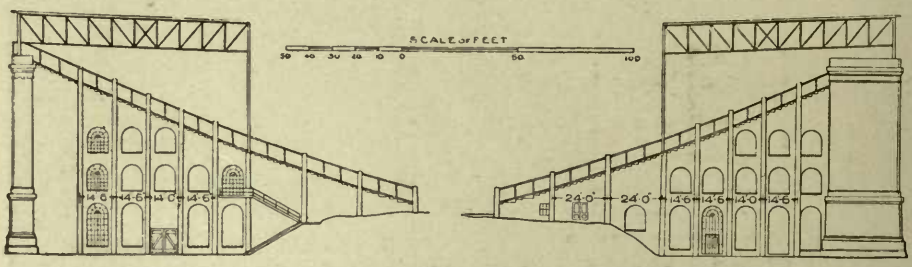


Fig. 653.—East and West Elevations of Grand Stand, St. Paul

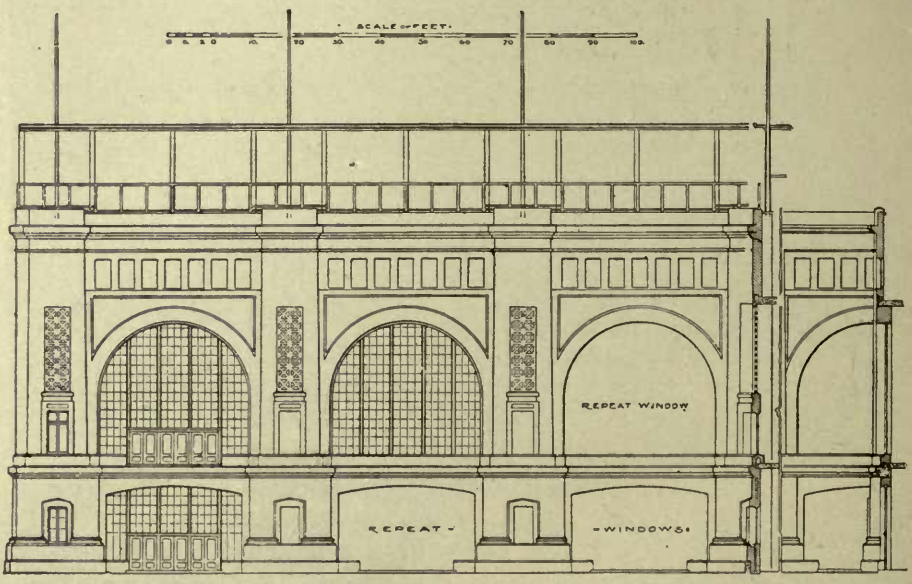


Fig. 654.—Half-elevation (to street) of Grand Stand, St. Paul

The method of finishing some of the surfaces is interesting. All the floor slabs were cast and finished at one operation, the concrete being made especially rich for the

purpose. The extra amount of floating had the effect of bringing more moisture than usual to the surface, and this sometimes caused shallow pitting, which, however, was not regarded as serious. Other exposed concrete surfaces were stippled with cement grout by dipping a bunch of stiff twigs in the grout and striking it sharply with a rod, causing the cement to spatter over the surface. In all cases where the surface had been thoroughly wetted, the cement adhered perfectly.

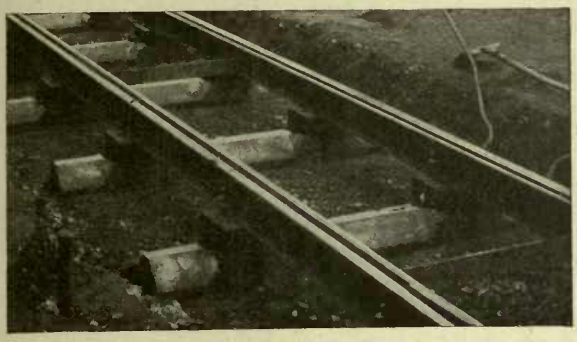


Fig. 655.—Railway Sleepers

RAILWAY SLEEPERS

The drawings (Fig. 656) and the photograph (Fig. 655) show reinforced concrete sleepers designed on the Italian model, and adopted in an experimental track laid between Dresden and Kotzschenbroda, Saxony. The reinforcement is clearly shown in the drawings, in which the U-shaped stirrup placed where the maximum shearing stress occurs will be noted. To receive the screw bolts there are wooden blocks of dovetail shape about 4 in. deep wholly embedded in the concrete except for the top surface. The sleepers are roughly 6 ft. long and weigh 209 lb., of which the reinforcement accounts for 10½ lb. The drawings show that the rails do not rest on the concrete; instead, the pressure is spread by means of an oak block about 3 in. high and 10 in. long. As illustrated, the rails are held by 7 in. screw bolts, but in a later system clamps are employed. At the end of a year, parts of the track were broken up for the purpose of closely examining the work, and the sleepers were found to be in perfect condition. The screws were tight, and the rails were not perceptibly corroded.

GRAIN SILOS AT SILVERTOWN

Reinforced concrete has proved itself to be an admirable material for the construction of silos, an excellent example of which is a granary erected at Silvertown on the Hennebique system, and consisting of eight cylindrical silos, each having a height of 80 ft. and a diameter of 20 ft., and having a storage capacity of 505 tons, the total capacity of the eight silos being about 18,000 quarters of grain. The silo walls are only 6 in. thick. The architect was Mr. F. E. L. Harris, A.R.I.B.A., and the contractors were Messrs. A. Jackaman and Son. The construction is clearly shown in the photograph and drawings (Figs. 657 to 664). Reinforced concrete piles to the number of 124, each 14 in. square in cross section, support a general foundation slab, immediately underneath which are tunnels or subways for the transport of the grain. Care was taken to see that the projecting reinforcements from the heads of the piles were well bonded into the columns, slabs, etc., so as to make the whole structure monolithic. With the same object in view the walls of the silos are connected one with the other, as clearly shown in the horizontal section, in which also the vertical ribs or counterforts to the outer walls of the silos will be noted.

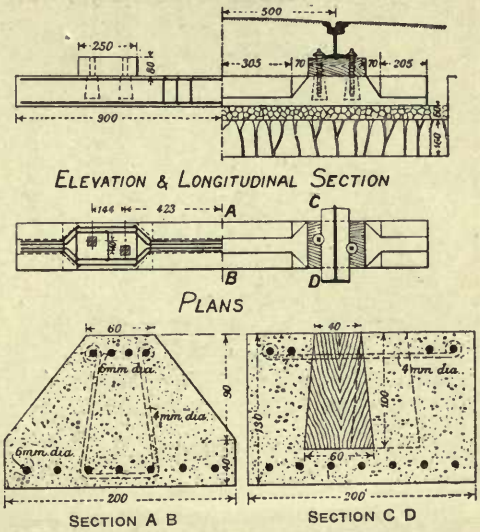
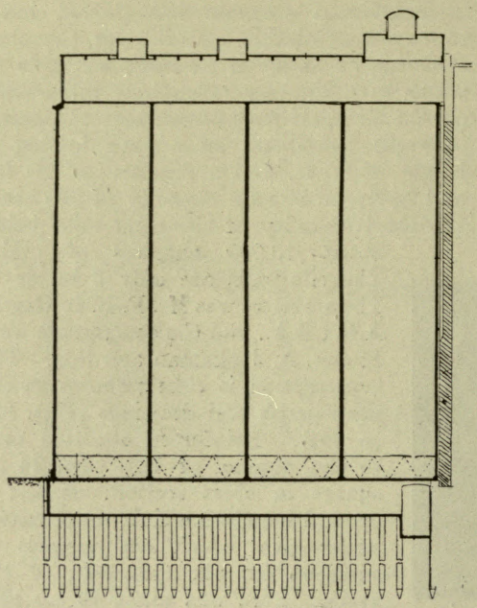
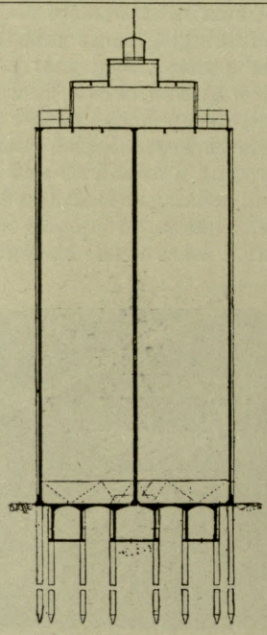


Fig. 656.—Details of Sleepers





Figs. 657 and 658.—Two Sections through Silos

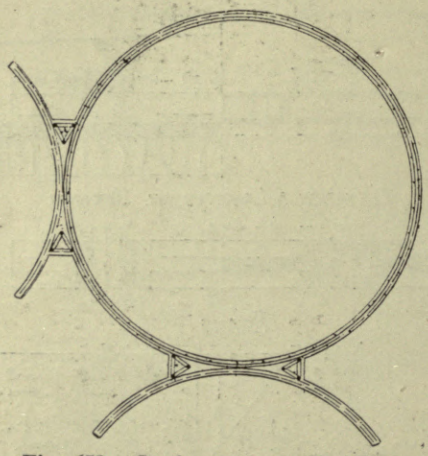


Fig. 659.—Section through Silo Walls

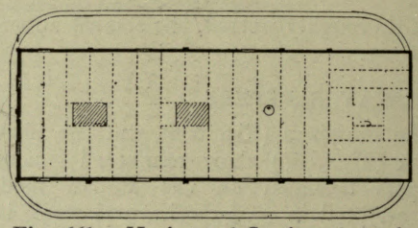


Fig. 661.—Horizontal Section through Silo Roof

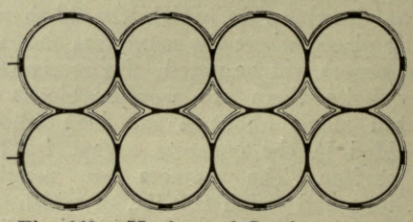


Fig. 662.—Horizontal Section through Silos (looking upwards)

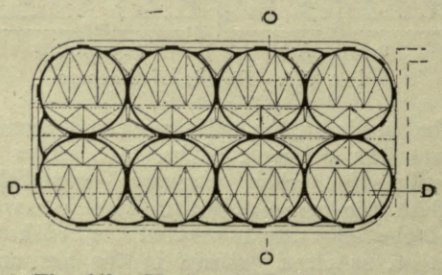


Fig. 660.—Horizontal Section through Silos

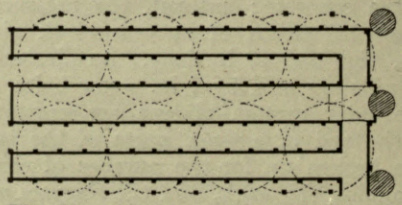
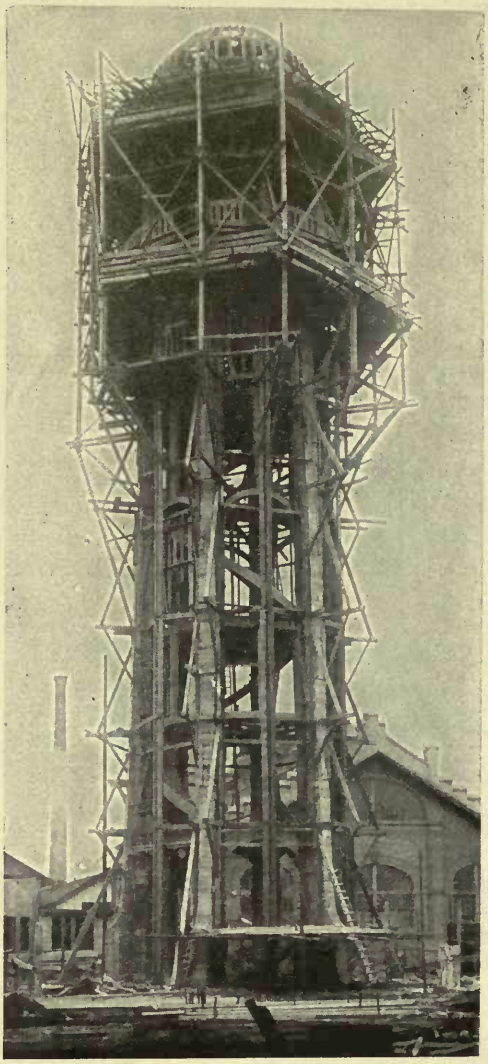


Fig. 663.—Horizontal Section through Silos, showing Tunnel Arrangement

WATER TOWER AT SINGEN

Figs. 664 and 665 are photographs, and Figs. 666 to 668 working drawings of a water tower erected in 1909, at a factory at Singen, Germany, the architect being H. Steub, and the designers and contractors being the firm of Josef Krapp, of Karlsruhe, whose engineer (F. Guske) has contributed to *Beton und Eisen* an article which has served as the basis for the following description. The total height is 141 ft., the base of the tank or reservoir being 98 ft. 5 in. above the ground. There are six columns, 6 ft. 6 in. by 1 ft. 8 in. in cross section at the base, tapering to 2 ft. by 1 ft. 6 in. at the top. The ring-shaped foundation has an outer diameter of 41 ft., and an inner diameter of 16 ft. 6 in. As shown in the vertical cross section, the floor of the tank is built in panel form, the panels being 9½ in. thick, and the ribs 2 ft. 3 in. thick. There are three floors below the tank level, the means of communication between them consisting of a spiral staircase carried on concealed corbels. Surrounding the tank is a gallery, also carried on corbels, which, in this case, are a feature of the elevation. The tank, 23 ft. 9 in. in diameter, and



Figs. 664 and 665.—Water Tower at Singen

21 ft. 8 in. high, is built of steel plate with a casing of movable slabs of pumice-stone

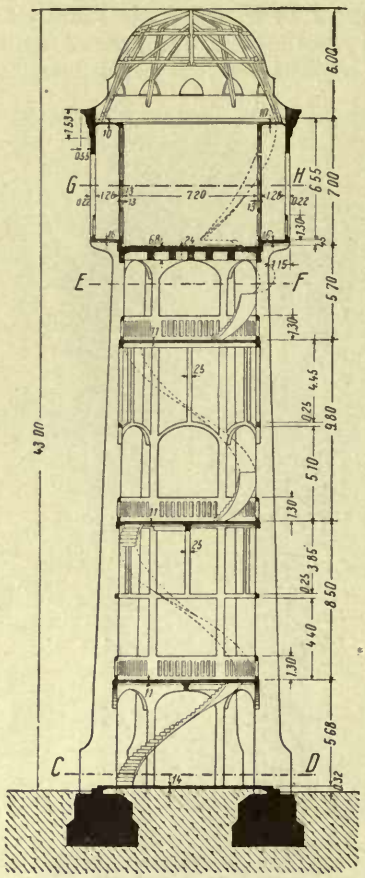
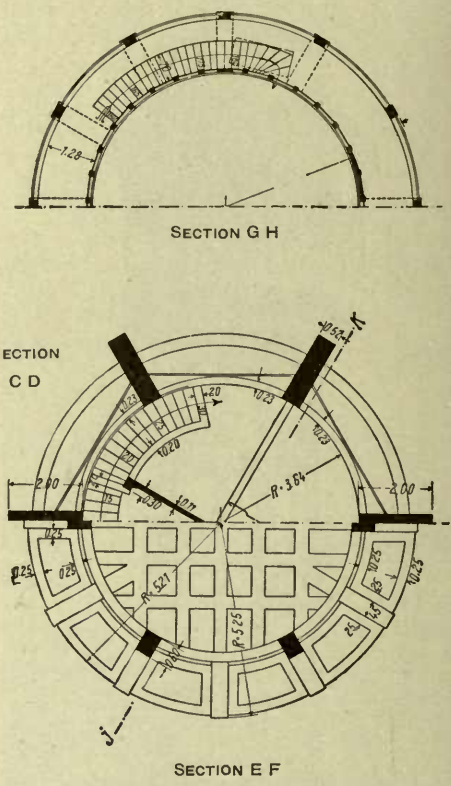


Fig. 666.—Vertical Section of Water Tower

concrete inserted between very thin vertical concrete supports, the space between the steel and the slabs being filled with dry peat. The roof is of wood, covered partly with slates and partly with copper. In designing the tower, a wind pressure of 30 lb. per square foot was allowed for; the load on the gravel soil is 23 lb. per square inch, and the greatest compressive strength in the structure is 467 lb. per square inch. On the completion of the work, water was run into the tank



Figs. 667 and 668.—Three Horizontal Half-sections of Water Tower

until a height of 20 ft. 4 in. was reached, the centre of the ribbed floor then showing a depression of 0·124 in. The dimensions indicated in all the drawings shown on this page are in metres and centimetres.

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